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REFLECTION CRACKING ON ASPHALTIC CONCRETE RUNWAY OVERLAYS IN COLD AREAS

by

© SHEUNG CHING POON

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
OF MASTER OF SCIENCE IN CIVIL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

FALL 1986

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Date..August 14/86..

ABSTRACT

Reflection cracking is a serious form of pavement distress which can shorten the service life of overlays and increase the maintenance cost of pavements. In this study, the reflection cracking problem in asphaltic concrete overlays of flexible airfield pavements in cold areas is examined.

The investigation firstly presents a review of various factors of asphaltic concrete which relate to reflection cracking at low temperatures. Following this review, a study of the fracture mechanism of an asphaltic concrete overlay and possible approaches for inhibiting the reflection cracks are then presented. Recent experience with different rehabilitation methods is presented, and the effectiveness of these methods in controlling the reflection cracking is discussed.

A test section on a Yellowknife, N.W.T. airfield runway constructed in 1983 is considered as a case study of the performance of the nonwoven polyester fabric (Mirafi P50 and P250) in inhibiting reflection cracking. Indirect tensile splitting tests were conducted on cored samples at temperatures -6.7 C, -17.8 C, -23.3 C and -28.9 C (20 F, 10 F, -10 F and -20 F). Asphalt cement recovered from the core samples was tested to determine its characteristics. ASTM standard and other tests included penetration at 25 C (77 F) and 4 C (39.2 F), kinemetic viscosity at 135 C (285 F), absolute viscosity at 60 C (140 F) and ring and ball

softening point.

A computer based cracking prediction model, the COLD program, was used to estimate the potential for cracking of the overlay in the winter of 1983-1984 which reached a minimum temperature of -42 C. The results were compatible with the observed field performance.

The literature indicates that reflection cracking can be reduced or eliminated by a number of rehabilitation methods. The more effective methods seem to be an open graded hot mix interlayer and an overlay of low consistency asphalts or modified asphalts. The fabric used in the Yellowknife project is not effective in resisting reflection cracking. Other rehabilitation methods are recommended for possible future projects.

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CHAPTER 1

INTRODUCTION

1.1 General

Cracking is one of the major distresses that occur in flexible (asphaltic concrete) highway and airfield pavements. Cracks formed in the pavement usually can be divided into load associated and non-load associated according to their formation mechanisms. Load associated cracks are caused by traffic stresses, and non-load associated cracks result from other than vehicular loads. The latter can be formed by thermally induced stress or by stresses due to foundation movements and moisture changes. Discussion of this particular form of distress was the subject of a symposium as part of the 1966 annual meeting of the Association of Asphalt Paving Technologists.

In cold regions, such as Canada and the northern part of the United States, non-load associated cracks are common in flexible pavements. Many of these are thermal cracks and are characterised by cracking in the transverse direction. Thermal contraction of the pavement during a long cold winter is the major cause for these transverse cracks. In airfield runway pavements, because of their relatively larger width, also develop fractures in the longitudinal direction and thus a blocky pattern is produced.

An extensive study of low temperature cracking has been reported by the Ad Hoc Committee of the Canadian Good Road

Association (Haas et al.,1970). While not necessarily judged as a primary determinant of low temperature cracking, the freezing index has been used to indicate potential areas for such cracking. However severe low temperature cracking have been reported for areas exposed to a freezing index of 560 C-days with some cracking also reported in areas with a freezing index of 360 C-days (Finn et al.,1976a).

Each year millions of dollars are spent in pavement maintenance and upgrading. Unrepaired cracks will further deteriorate by spalling and develop into pot holes, resulting in poor riding quality. Riding quality is much more critical in airfield runways than that in highways. In airfield runways, a certain degree of smoothness must be provided for high-speed operations occurring during landing or take off. Loosened particles from spalled cracks also may cause damage to aircraft when these particles are subjected to blast by aircraft engine exhaust. Crack sealing can minimize the spalling problem but it cannot prevent the riding quality of a pavement from deteriorating.

As the asphaltic concrete pavement ages with time, additional cracking and other distresses such as fatigue - type cracking and rutting also may become more serious, consequently the serviceability of the pavement decreases. Level of pavement serviceability is determined according to the pavement's structural capacity, riding comfort, distress and skid resistance. When the pavement serviceability reaches the minimum desirable level, pavement rehabilitation is required.

The need for rehabilitation is invariably satisfied by overlaying the existing pavement with another asphaltic concrete layer. Unfortunately, cracks in the pavement that have been sealed and overlaid reappear shortly after the overlay. This type of premature cracking is commonly known as reflection cracking since it reproduces the same crack pattern as the underlying old pavement. This thesis will investigate possible rehabilitation methods with a view to minimizing this reflection cracking problem.

1.2 Objectives of The Thesis

The primary objective of this research is to study the reflection cracking problem in asphaltic concrete overlays of flexible airfield pavements in cold areas. Literature dealing with rehabilitation methods has been reviewed in order to identify possible techniques to reduce, if not completely eliminate, this problem.

A secondary objective is to review a case study of an asphalt concrete runway overlay constructed in Yellowknife, N.W.T. in 1983. As a research assistant on this project, the author was involved during the construction of a test section in which a fabric was used as an interlayer prior to placing the asphaltic concrete overlay. This thesis undertakes a further investigation of this test section.

1.3 Scope of The Thesis

Recent North American literature published within the past decade dealing with reflection cracking has been selected as the main source of information. Related experience from Japan has become available through the Canada/Japan Paving in Cold Areas (PICA) workshops held in 1982 and 1984. The proceedings of these conference have been studied as a source of information in the reflection cracking problem.

Further evaluation of the Yellowknife overlay project of 1983 has been undertaken. Tests on cores taken at the time of construction have been used to assess the potential for cracking of the overlay test section. Subsequent pavement condition surveys have been used to evaluate the test section after the severe winter of 1983-1984.

1.4 Organization of The Thesis

The first chapter gives the introduction along with the objectives and the scope of the thesis. The organization of the thesis is also included.

The second chapter reviews the basic characteristic of asphalt and its relationship with time and temperature. The cracking mechanism and the criteria of cracking are studied. Finally, different rehabilitation methods are categorised according to the fracture mechanism.

Published literature and reports about experimental projects done in Japan, the United States, and Canada are reviewed in the third chapter. The methodologies, the development and the problems, as well as the experimental results are discussed.

The fourth chapter discusses the test project done in Yellowknife, N.W.T. in 1983. Important aspects during the construction are described. The performance of the test section after the winter of 1983-1984 is evaluated in this chapter.

The fifth chapter describes the testing program for the cored samples of the Yellowknife Project and the computer analysis to study the potential for cracking of the overlay under the critical temperatures of winter 1983-1984.

The sixth chapter presents and analyzes the results of the test and the computer analysis.

The seventh chapter is the summary, conclusions and recommendations of this investigation. It also includes future research work suggested for further studying the problem.

CHAPTER 2

SELECTIVE REVIEW ON VARIOUS FACTORS RELATED TO REFLECTION CRACKING ON FLEXIBLE PAVEMENT OVERLAYS AT LOW TEMPERATURES

2.1 Introduction

Extensive studies of transverse cracking of the pavement date back to 1960's. Field evidence of the different capability of various asphaltic concretes to resist slow tensile strains without cracking was presented by Shields and Anderson (1964). They showed several examples of pavements where large differences in transverse cracking were evidenced between pavements whose principal difference appeared to be the source of the asphalt used in the asphaltic concrete. Similar finding were also reported by Culley (1966), McLeod (1969) and Shields et al. (1969). These researchers, including Young et al. (1969) and Haas et al. (1970), have found that different grades of asphalt exhibited significantly different crack frequencies in the pavement surface. In view of this, a review of various physical characteristics of asphalts and asphaltic mixes and their relationship to the rupture mechanism has been undertaken. Attempts to apply basic fracture mechanics principles in order to develop appropriate rehabilitation techniques are also described in this chapter.

2.2 The Low Temperature Behaviour of Asphalt

Asphalt is a viscoelastic material, and this viscoelastic behaviour changes with temperature.

Heukelom (1966) divided the strain of asphalt into three characteristic parts, namely the elastic strain, the viscous strain, and the delayed-elastic strain. Their relationships with the stiffness modulus are shown in Fig.2.1. He also stated that at very high stiffness values, thus at extremely low temperature, the strain of asphalt became mainly elastic.

Similar results were reported by Haas and Anderson (1969) in their study of tension tests on thin films of asphalt over a range of temperatures. Fig.2.2 shows that as temperature decreases below 0 C, asphalt changes from viscoelastic to more elastic in nature; and at very low temperature, it is generally elastic.

2.3 Stiffness of Asphalt and Asphaltic Mix

The stiffness modulus of an asphalt has the same dimension as Young's modulus except that it varies as a function of loading time and temperature.

$$(S)_{t,T} = \left(\frac{\sigma}{\epsilon} \right)_{t,T} \qquad (2.1)$$

The stiffness modulus of an asphalt at a particular loading time and temperature depends very much on the grade of the asphalt and its temperature susceptibility. The temperature susceptibility of an asphalt is the temperature rate of change of consistency of an asphalt.

Penetration at low temperatures and viscosity at high temperatures are measures used to define asphalt consistency. Pfeiffer and Van Doormaal (1936) found that when the logarithm of the penetrations were plotted against the test temperatures, an almost straight line was obtained, and the slope was considered as a measure to the temperature susceptibility of the asphalt. They also found that for a number of asphalts, the penetration value at ring and ball softening point was approximately 800. Since the ring and ball softening point is determined routinely for many asphalts, Pfeiffer adopted it conveniently as one of the two test values needed in evaluating the temperature susceptibility. The equation for temperature susceptibility is expressed as

$$A = \frac{\log 800 - \log \text{pen at } T}{T_{R\&B} - T} \qquad (2.2)$$

Pfeiffer and Van Doormaal also introduced the penetration index (PI) as a scale to measure the temperature susceptibility in such a way that a Mexican bitumen with a penetration of 200 at 25 C gives a PI of zero. The expression for PI is

$$\frac{20 - PI}{10 + PI} = 50\ A$$

$$\text{or} \quad \frac{20 - PI}{10 + PI} = 50\ \frac{\log 800 - \log \text{pen}}{T_{R\&B} - T} \qquad (2.3)$$

A nomograph presented by Van der Poel (1954) to find the PI value of an asphalt is shown in Fig.(2.3) He also developed a nomograph to predict the stiffness of an asphalt (Fig.2.4). The limitation of these nomographs is that they can not be used with asphalts containing more than two percent of wax by weight.

Kopvillem et al. (1969) found that Eqn.(2.3) was not valid for all asphalts. For more complicated asphalts, such as blown asphalts and waxy asphalts, the penetration at $T_{R\&B}$ might not be equal to 800. For blown asphalts, the temperature susceptibilities above the softening point do not correspond with their PI values below the softening point. While for waxy asphalts, these PI values might not be different, however, the viscosity of a waxy asphalt was considerably lower than that of an equal graded ordinary asphalt. To resolve the above mentioned discrepancy, the authors developed the Shell Method --- a Bitumen Test Data Chart (BTDC) to determine the PI value of an asphalt. In view of the departure from 800 pen at $T_{R\&B}$, temperature T_{800}, taken at 800 pen, was used instead. The BTDC for Class S (straight-run) asphalt, Class B (blown) asphalt and Class W (waxy) asphalt is shown in Fig.2.5.

It is apparent that a transition range exists between the two straight branches of the curve of a waxy asphalt. Within the transition range, wax which is crystallized in the asphalt at low temperatures begins to melt and thereby affects the consistency of the asphalt. The standard penetration test taken at 25 C, a method often used as a routine test to indicate the grade of a bitumen, is also affected by this transition range. Heukelom (1969) found that the wax, once it had crystallized, remained in the crystalline form up to fairly high temperatures. A pre-cooling procedure, i.e. to cool down the asphalt to 0 C and then heating up to the temperature of the penetration test, provides a more accurate method to measure the penetration value and thus the properties of the asphalt.

The Shell Method requires the measurement of penetration at all temperatures using 100 g loading for 5 sec. Schmidt (1975) developed a nomograph to find the PI of an asphalt using viscosity at 60 C and penetration data obtained in ASTM tests, i.e. penetration at 25 C, 100 g loading for 5 sec, and penetration at 4 C, 200 g loading for 60 sec. A report from The Asphalt Institute (1981) commented that this procedure is less desirable when compared to the method using penetrations measured under the same loading conditions, however there is some merit to Schmidt's nomograph when only ASTM penetration data are available.

McLeod (1976) introduced the Penetration-Viscosity Number (PVN) to substitute for the PI in defining the temperature susceptibility of asphalts. Instead of the penetration at the ring and ball softening point used in PI, viscosity in centistokes at 135 C was used (Fig.2.6). It was suggested that the PVN derived from this method gave the same value as the PI of an ordinary asphalt. The Asphalt

Institute report (1981) commented that McLeod's PVN system is reasonable for selected non-waxy, non-air-blown asphalts only. When applying this method to asphalt with wax content in excess of two percent and air-blown asphalts, error is expected.

When the PI or PVN of an asphalt is known, the stiffness of an asphalt can be determined using Van der Poel's nomograph or the nomographs modified by Heukelom and McLeod respectively. The corresponding stiffness of an asphaltic concrete can also be determined using Fig.2.7.

The stiffness of an asphaltic concrete can also be estimated by direct testing. Numerous test methods have been reported, and each method has its particular advantages and disadvantages. Among these tests are creep test, relaxation, constant-rate-of-extension, tensile splitting test, dynamic and repeated axial or flexural tests. The tensile splitting test has been used extensively in the analyses of cracking in Alberta. This test method is used to obtain the tensile strength - temperature relationship for the asphaltic concrete of the Yellowknife Project. Details of the test will be described in Chapter 5.

2.4 Time and Temperature Equivalency Hypothesis of Asphalt

Asphalt is suggested to be a 'thermorheologically simple' material, meaning that the effect of loading time on the stiffness of the material can be expressed by the effect of temperature change. Monismith et al.(1965) showed that if the change of the stiffness of an asphalt over time at a series of temperatures were plotted in a stiffness-log time graph, a family of similar shape response curves would be obtained (Fig.2.8). The response curves can be shifted along the logarithmic time axis to a selected reference temperature T_0, and the temperature equivalent time or the 'reduced time' for the shift is determined as

$$\xi = t \exp\{f(T)\} \quad (2.4)$$

where ξ is the reduced time corresponding to a real time, t, at temperature T_0

and $f(T)$ is the temperature function giving the shift of the response curve.

The temperature function f(T) is generally termed the shift factor, and is equal to the ratio of time for testing temperature T to have a specified mix stiffness to the time for reference temperature T_0 to have the same mix stiffness. With the 'reduced time' function, the rheologic response of an asphaltic concrete over a long time range can be conveniently defined.

2.5 Relationship of Stiffness to the Fracture Susceptibility of Pavement

The stiffness of an asphaltic concrete is directly related to the fracture susceptibility of the asphaltic concrete pavement. Sugawara et al.(1982) showed in a nomograph (Fig.2.9) that the softening point, the penetration and the PI value, which are variables of the mix stiffness, had a direct relationship to the fracture temperature of the mix. The nomograph is said to be valid for asphaltic concrete with volume concentration of aggregate C_v = 0.86 to 0.84.

McLeod (1969) has concluded that, on the average, transverse cracking would occur if the stiffness of the asphaltic concrete, at the minimum service temperature encountered, fell within the range of $1x10^6$ to $2x10^6$ psi. ($6.9x10^9$ Pa. to $1.4x10^{10}$ Pa.). These asphalt stiffness values were derived from McLeod's 1969 modification of Van der Poel's nomograph at a loading time of 20,000 seconds. Fromm and Phang (Haas, 1973) suggested that the critical asphalt cement stiffness was 20,000 psi. ($1.4x10^8$ Pa.) for 10,000 seconds loading time.

With a more sophisticated method, Gaw (1978), based on data from the Ste. Anne Test Road, reported that an asphalt cement stiffness value of $1x10^9$ Pa. at a loading time of 1800 seconds was the limit above which cracking would occur.

Using a lower stiffness asphaltic concrete can lower the fracture temperature of the asphaltic concrete pavement and therefore pavement can withstand a harsher winter without cracking. However, adequate stiffness has to be maintained for the stability of the pavement to prevent the pavement from rutting. It is the goal of many investigators to find a solution to this dilemma.

2.6 Fracture of Asphaltic Concrete

Cracking of an asphaltic concrete overlay occurs when the overlay stresses, either externally applied or internally developed, exceed the tensile strength of the material. External applied stresses are those due to traffic load and subsurface movement, while internally developed stresses are mainly the thermally induced stress in considering cracking of overlays in cold areas.

2.6.1 Thermally Induced Stress

Thermally induced stress is a function of the rate of cooling, the coefficient of thermal contraction and the stiffness of the mix.

Assuming temperature dependent viscoelastic behaviour and 'thermorheologically simple' behaviour to asphaltic concrete, Humphreys and Martin (1963) expressed the hydrostatic tensile stress for an asphalt concrete slab as:

$$\sigma_x(z,t) = -3a_o \int_0^t R\{\xi(z,t)-\xi(z,\tau)\} \frac{\partial}{\partial\tau} \theta(z,\tau)d\tau \qquad (2.5)$$

where z = depth

t = time

a_o = coefficient of thermal expansion/contraction at reference temperature T_0

ξ = the reduced time as introduced in Section 2.4

τ = time integration variable

θ = the pseudo-temperature

R = function of relaxation moduli of the material

Monismith et al.(1965) simplified Eqn.2.5 using a constant coefficient of expansion/contraction. In order to have the equation evaluated numerically by a computer, the equation was further modified using the trapezoidal rule. The resulting equation is

$$\sigma_x = -\frac{3\,\alpha}{2} \sum_{i=1}^{N} \{R(\xi-\xi_i)+R(\xi-\xi_{i-1})\}\,(T_i-T_{i-1}) \qquad (2.6)$$

where N is the number of time steps from 0 to t.

On the other hand, assuming a pseudo-elastic behaviour to asphaltic concrete, Christison (1972) expressed the stress equation for slab analysis as

$$\sigma_t = -\int_{t_0}^{t} S(\Delta t,T)\,\frac{\alpha_o(T)}{1-\nu(T,t)}\,dT(t) \qquad (2.7)$$

where $S(\Delta t,T)$ = time and temperature dependent stiffness modulus.

Eqn. 2.7 becomes simple elastic if both S, ν and α are assumed to be independent of time and temperature.

Christison (1972) employed five different methods in the computation of thermal stresses in asphaltic concrete pavements. These methods are:

1) pseudo-elastic beam analysis
2) approximate pseudo-elastic slab analysis (assume ν is constant)
3) viscoelastic slab analysis
4) viscoelastic beam analysis
5) approximate viscoelastic slab analysis. (assume ν is constant)

The suitability of these stress analyses were compared by correlating a) times of predicted initial cracking with observed times of initial cracking and b) the time periods during which subsequent cracking was observed with the time periods in which the predicted stress exceeded the fracture criterion employed. The results showed that the viscoelastic slab and beam analyses appeared to overestimate and underestimate the stresses respectively. The pseudo-elastic beam, the approximate viscoelastic slab and the approximate pseudo-elastic slab methods yielded reasonable intermediate values of stresses (Christison et al.,1972). A computer based prediction model for low-temperature cracking developed by Christison using the pseudo-elastic beam analysis was modified so as to arrive at the damage prediction model referred to as COLD, (Computation of Low-Temperature Damage) under the National Cooperative Highway Research Program Project 1-10B by Woodward-Clyde Consultants, San Francisco (Finn et al., 1976b,1977). This program was used in the analysis presented in Chapter 5.

Sugawara and Moriyoshi (1984) reported the study of the effect of mix composition and cooling condition on the thermally induced stress of asphaltic concrete. Little difference in the thermally induced stress was found by changing the asphalt content in the mix. However changes in the percentage of air voids in a mix showed considerable difference in the thermally induced stress. Fig.2.10 shows that lower thermally induced stress was obtained in mix of higher air voids and higher thermally induced stress was obtained in mix of lower air voids. Fig.2.11 shows that the thermally induced stress would decrease with time, even though the specimen was maintained at a constant temperature. It has been suggested that growth of micro cracks during stress relaxation might cause the specimen to crack at a temperature lower than the expected fracture temperature. The resemblance of the stress-time lines between the second cooling and the first cooling shown in Fig.2.12 and Fig.2.13 indicate that the thermally induced stress of asphaltic concrete depends only on the temperature change and is not affected by the preceding condition. Fig.2.12 also indicates that there are no significant differences between the fracture temperature of the mixes before and after the stress relaxation. Fig.2.13 also shows complex stress conditions exists in cyclic warming and cooling of asphaltic concrete, with some compressive stresses develop during the warming phases.

2.6.2 Subsurface Movement Induced Stress

Stress can be induced in the attached overlay by the movement of the underlying old pavement. Pavement movement may be caused by one or a combination of the following factors.

1. Temperature Change

   Temperature change causes thermal expansion or contraction of the underlying pavement and subgrade. This volume change results in the undesirable pavement movement.

2. Moisture Loss in Subgrade

   Through moisture loss, shrinkage of the subgrade occurs and pavement movement develops.

3. Traffic Loading

   Vehicular loads or tractional forces produce shearing and tearing stresses in the pavement. The damage is aggravated by voids existing in the subgrade. Voids can be created by surface water which washes out fine materials through the cracks or by 'pumping' action.

4. Seasonal freezing and thawing

   Seasonal frost heaving and thaw consolidation of the underlying foundation can result in volume change and thereby induce stress in the overlaying pavement layer. Carpenter et al.(1975) have examined the freeze-thaw activity of the base course as an important mechanism in pavement cracking.

   Seasonal thawing also weakens the bearing capacity of the supporting embankment. Pavement supported by this embankment will be more vulnerable to the traffic load.

2.6.3 Tensile Strength of Asphaltic Concrete

Tensile strength of an overlay is a function of temperature, time, stiffness of mix, aggregate and the gradation of the mix. The tensile strength of an asphaltic concrete can be obtained either by experimental methods, such as a direct tension test, or by estimation methods.

Heukelom (1966) has indicated that the fracture of an asphaltic concrete is generally caused by the fracture of the asphalt cement. The stress and strain in the asphalt cement can be assumed as directly proportional to the stress and strain applied to a given mix. The multiplication factor depends upon the asphalt content of the mix, the mix gradation, the degree of compaction and the mineral composition of the aggregate. This factor can be defined as the ratio of the tensile strength of the mix to that of the asphalt cement and is assumed to remain constant for a given mix under all conditions of loading time, rate of deformation and temperature. Heukelom also found, with the aid of tensile tests, that the tensile strength of asphalt cements is a unique function of their stiffness. Using the above described function and the multiplication factor, the values of tensile strength of asphaltic concretes can be estimated from the stiffness of asphalt cement (Hills and Brien, 1966).

2.7 Rupture Mechanism

In view of the crack pattern of reflection cracking, it is believed that the formation of reflection cracks is directly related to the presence of the cracks in the old pavement. Pavement cracks destroy the continuity of the pavement structure, and produce a stress-concentrating effect in the overlay. Besides, under the pressure of wheel load, the pavement structure has maximum deflection at the crack. Thus, bending stresses in the overlay are also maximized in these areas. As induced stresses develop in the overlay, and with the old crack as a crack initiation, the crack propagates directly upward into the overlay. This explains why cracks of the overlay are mostly aligned with the old cracks of the underlying pavement.

The propagation of a crack may occur in the following three ways named according to the affecting stresses: the tension opening (type I), the shear opening (type II), and the tear opening (type III) (Fig.2.14). A decrease in temperature and other volumetric changes are generally the cause for type I opening, while vehicular loading are generally the cause for type II and type III openings. Tension opening occurs when the two sides of the old cracks of the underlying pavement move apart perpendicularly from one another causing tension at the crack tip. Shear opening occurs when the two sides of the old crack displace vertically with respect to one another causing shear force at the crack tip. Tear opening occurs when the two sides of the old cracks shift sideway one from another inducing shear force in a direction parallel to the crack tip (Monismith et al.,1980 and Langlois,1984).

2.8 Suggested Treatment Methods

A symposium dealing with reflection cracking treatment methods was part of the 1980 annual meeting of the Association of Asphalt Paving Technologists. In the symposium, a number of new strategies available to reduce the problem were discussed, and these provide a source of information to aid in understanding and applying techniques to mitigate reflection cracking.

Monismith et al.(1980) reported the use of the crack arrest principle in an attempt to solve the reflection cracking problem. In his explanation, Monismith considered the stress distributions for an infinite elastic plate containing an elliptical crack, as shown in Fig.2.15, in order to provide some insight to a possible crack arrest mechanism. His discussion was as follows:

> The uniaxial tensile stress S on the plate normal to the crack plane can be assumed to be similar to thermal contraction stresses acting in an overlay. From the figure it can be seen that a tensile stress σ_x is induced on a plane normal to the crack plane in front of the crack tip. If now the material ahead of the crack is anisotropic or orthotropic such that the stress σ_x exceeds the strength in the x direction while σ_y does not exceed the strength in the y direction, then the crack tip could be redirected in a direction normal to its original plane.

Based on this principle of crack arrest, reflection cracking rehabilitation techniques may be categorized in the following ways:

a) Elimination of Crack Tips

It is known that pavement cracks do not occur in elliptical shapes. In fact they are Y-shaped with a wide opening at the surface and reduce sharply to a flaw as they penetrate down into the pavement. A suggested rehabilitation method is to remove the top part of the Y and to seal up the flaw of the bottom part. An example of this is the Heater Scarification method.

b) Redirection of the Crack

As was explained previously in the quotation from Monismith about the crack arrest principle, the growth of the crack tip could be redirected in a direction normal to its original plane if the material ahead of the crack is anisotropic or orthotropic. A schematic representation of a possible sequence for this mechanism is given in Fig.2.16. A similar mechanism is typically the case when a stress relieving interlayer is provided between the old pavement and the overlay. Stresses induced by pavement movement are released by the delamination of the overlay from the old pavement. However, if this delamination continues, the overlay may become completely separated from the pavement and can be easily broken under the tractive forces of traffic, particularly if it is a thin overlay. An example of this rehabilitation method is a stress relieving interlayer using a geotextile.

c) Alteration of the Overlay Material

As mentioned in section 2.5, the stiffness of an asphaltic concrete is directly related to the fracture susceptibility of the overlay. Therefore altering the stiffness of the material used in an overlay is another method to reduce reflection cracking. The material chosen should have a stiffness that can withstand large deformation without rupture. Overlays using soft asphalts, rubber asphalts, or sulfur asphalts are examples of this technique.

d) Reinforcement of the Asphaltic Concrete

Increasing the tensile strength of an asphaltic concrete by reinforcing with a high tensile strength material is another rehabilitation method. Asphalt reinforcement such as TENSAR, fibreglass wiremesh and steel wiremesh belong to this type of technique.

2.9 Summary

From the studies reported, the stiffness of the asphalt cement and the asphaltic concrete is a primary factor of the fracture susceptibility of an overlay. Stiffness of an asphalt is a time and temperature dependent property. The effect of loading time and temperature can be superimposed one to another.

The stiffness of an asphalt cement and an asphaltic concrete can be estimated by direct testing or by indirect methods using nomographs. There is a significant deviation in the rheology of blown asphalts and waxy asphalts from ordinary asphalt. Modified methods to determine the stiffness of blown asphalts and waxy asphalts are also described.

Both the tensile strength and the induced thermal stress of an overlay are functions which depend on the stiffness modulus of the asphaltic concrete. Cracking of an overlay occurs when the induced stresses are greater than the tensile strength of the asphaltic concrete. Investigators also found that, on an average, cracking occurs if the stiffness of the asphalt cement and the asphaltic concrete is higher than a certain critical value.

Cracking mechanisms and fracture mechanics were also briefly reviewed in this chapter. Rehabilitation methods based on the fracture mechanics principle as well as examples of current rehabilitation techniques were also presented. The experience with these rehabilitation methods will be discussed in Chapter 3.

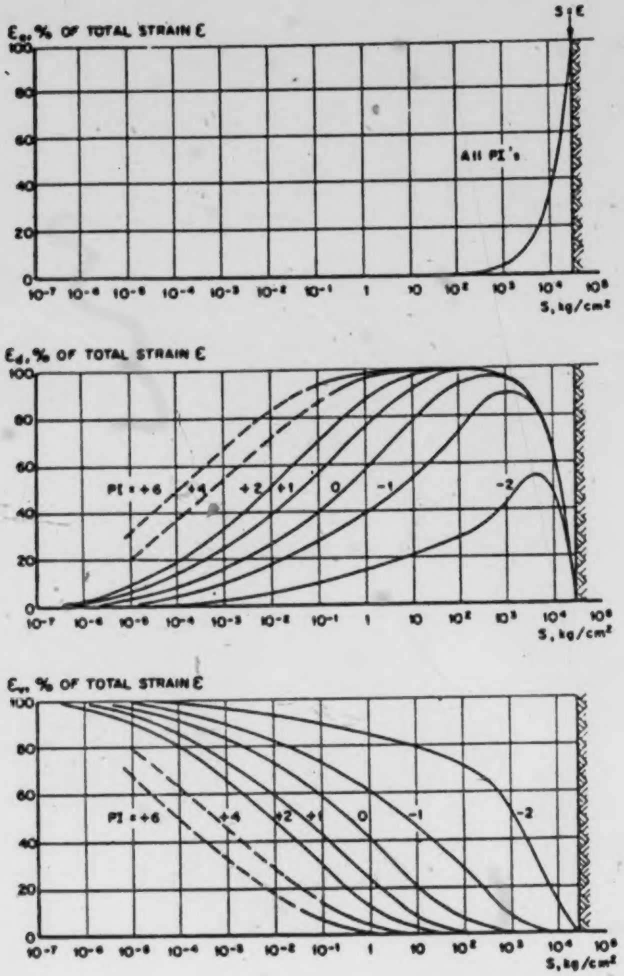


FIG. 2.1 THE RELATION OF ELASTIC STRAIN E$_e$, DELAY-ELASTIC STRAIN E$_d$, AND VISCOUS STRAIN E$_v$ TO STIFFNESS MODULUS (After Heukelom,1966)

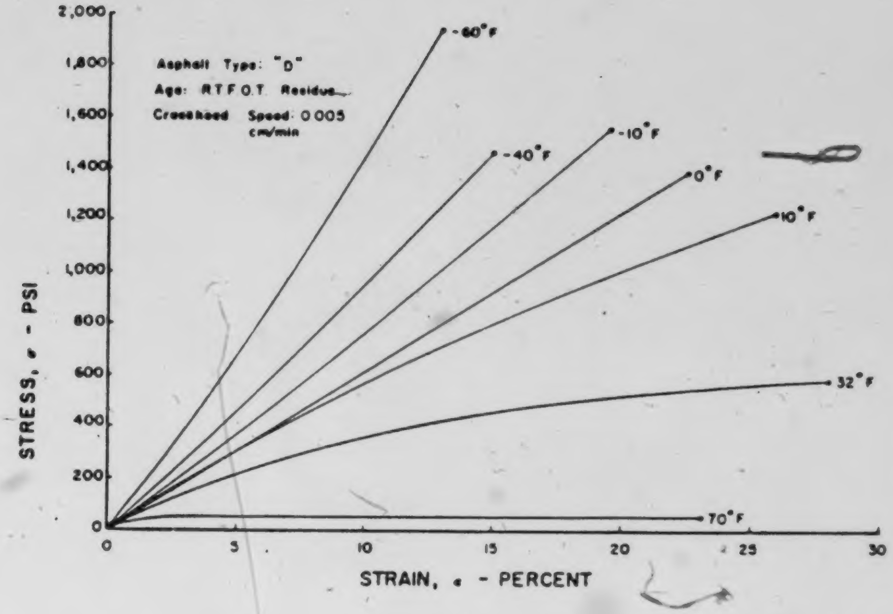


FIG. 2.2 THE TEMPERATURE EFFECT ON THE STRESS-STRAIN CURVE OF AN ASPHALT (After Haas and Anderson,1969)

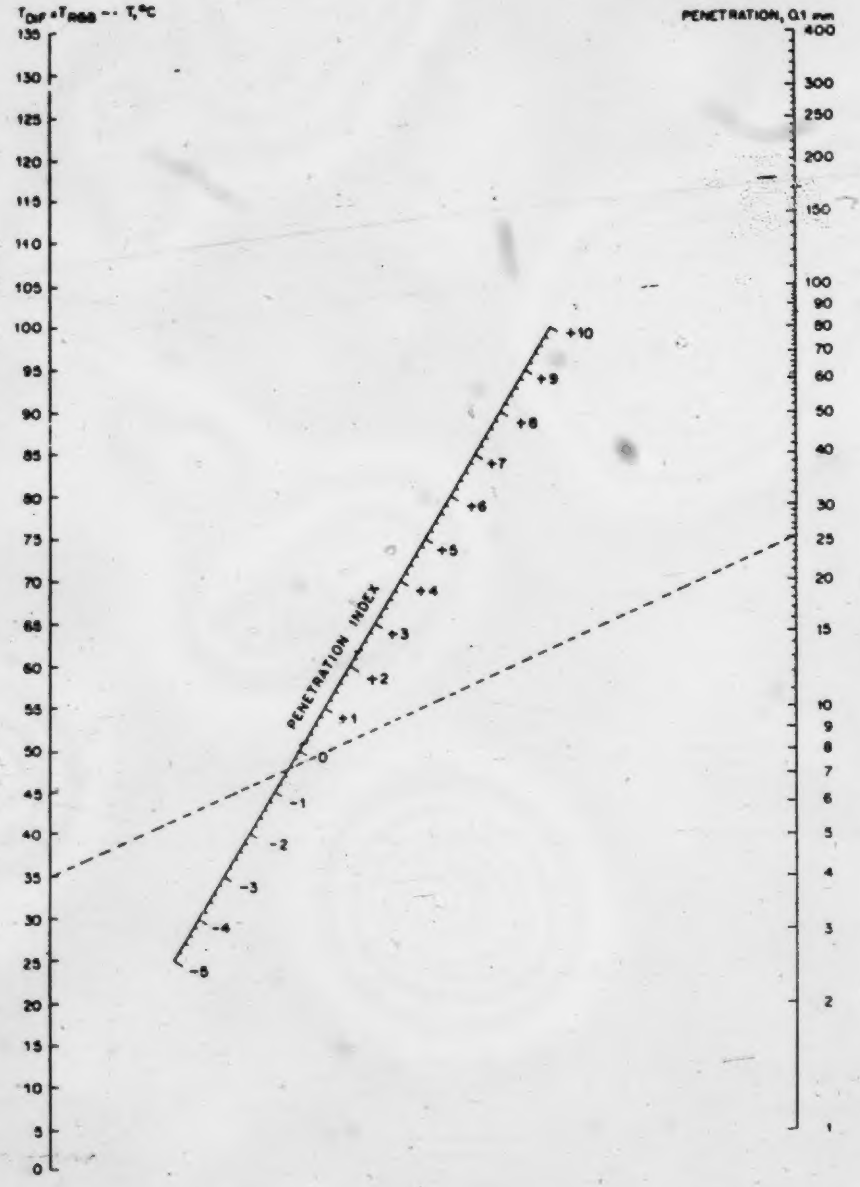


FIG. 2.3 DETERMINATION OF THE PENETRATION INDEX ACCORDING TO PFEIFFER AND VAN DOORMAAL'S NOMOGRAPH (After Van der Poel,1954)

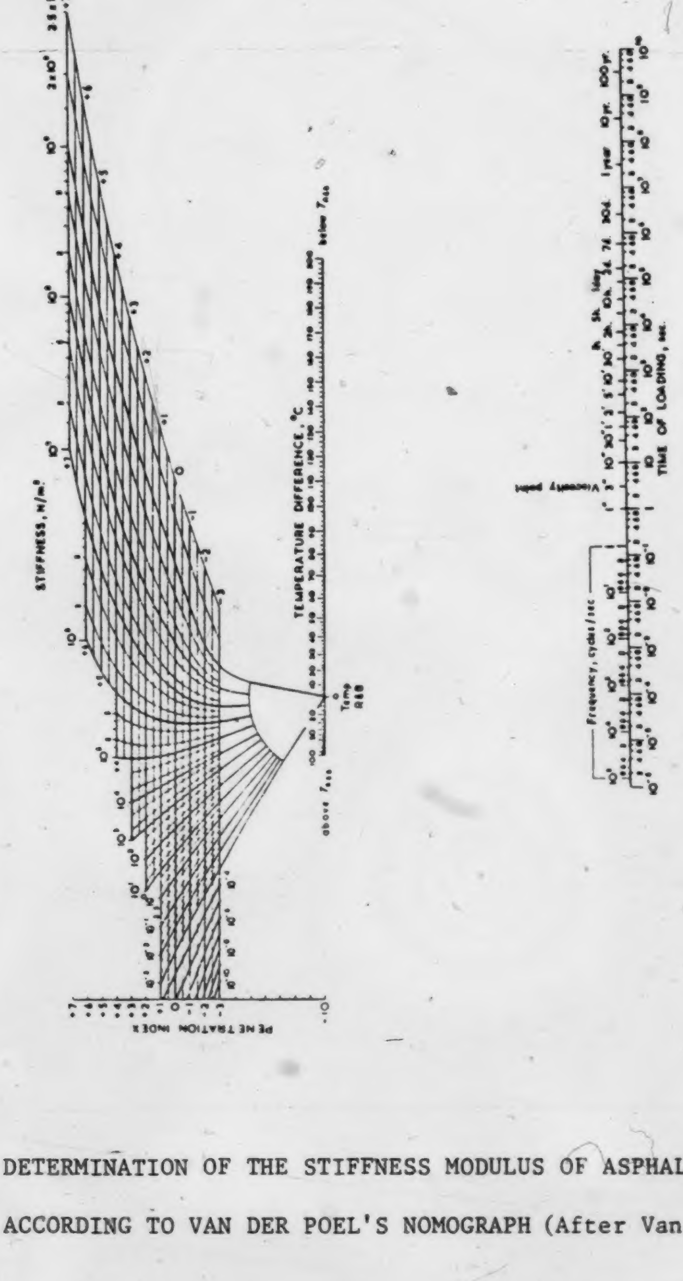


FIG. 2.4 DETERMINATION OF THE STIFFNESS MODULUS OF ASPHALTS ACCORDING TO VAN DER POEL'S NOMOGRAPH (After Van der Poel,1954)

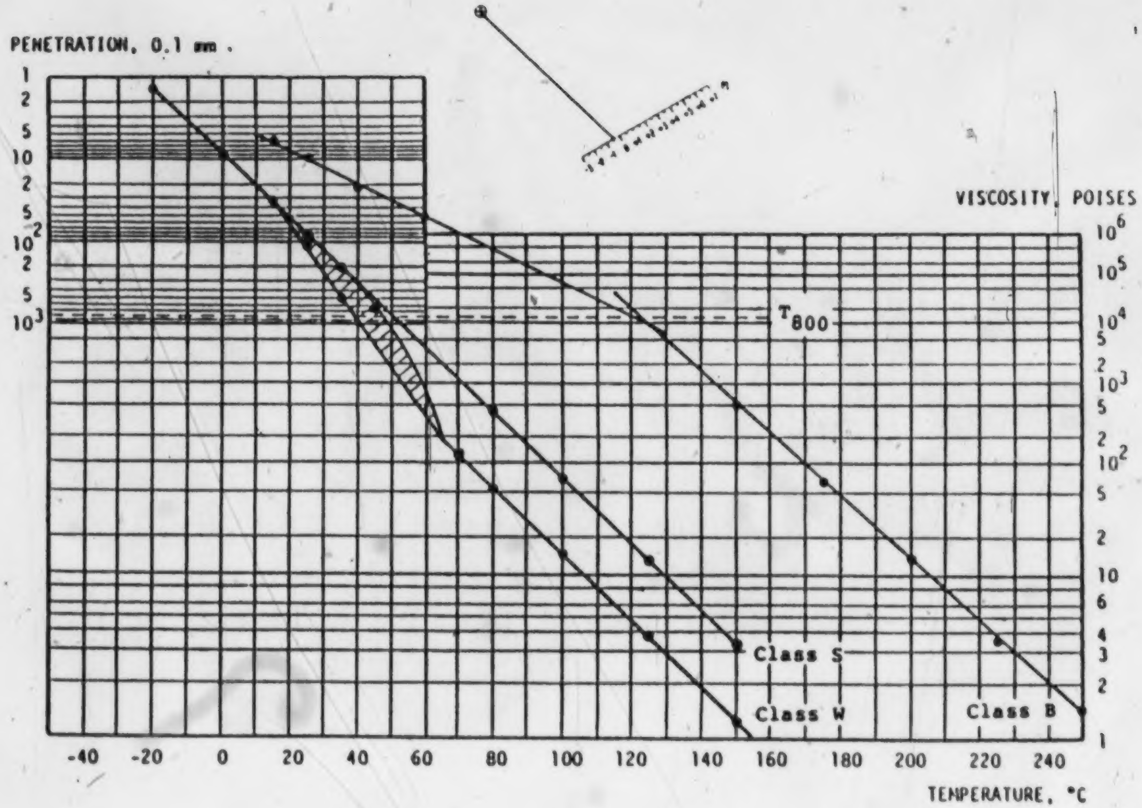


FIG. 2.5 THE BITUMEN TEST DATA CHART FOR CLASS S (STRAIGHT-RUN), CLASS B (BLOWN) AND CLASS W (WAXY) ASPHALTS (After Kopvillem and Heukelom,1966)

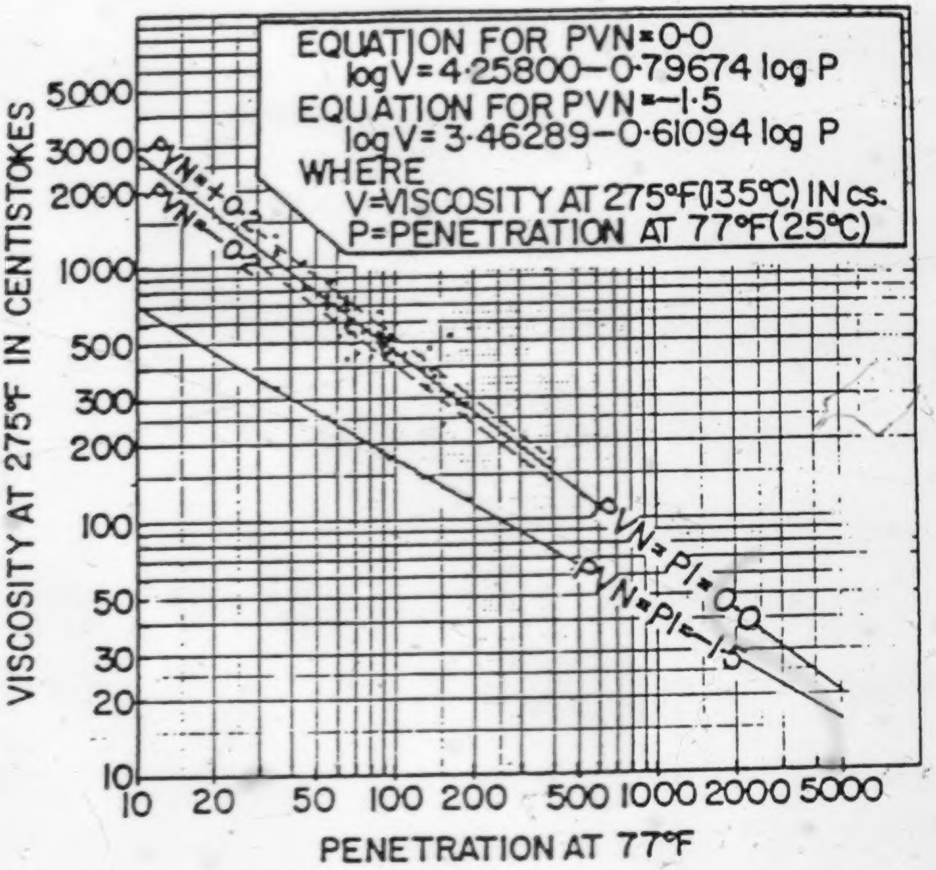


FIG. 2.6 DETERMINATION OF PEN-VIS NUMBER ACCORDING TO MCLEOD'S METHOD

(After McLeod,1976)

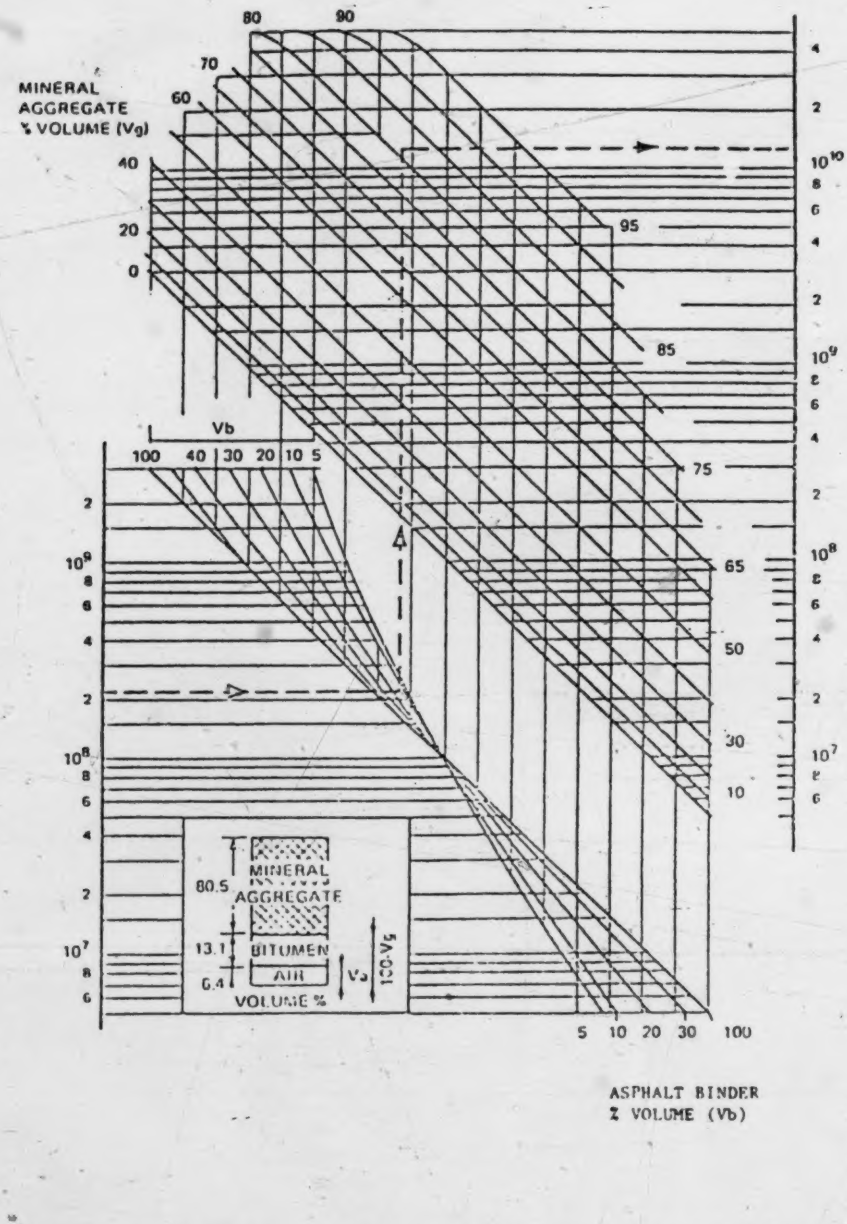


FIG. 2.7 DETERMINATION OF THE STIFFNESS MODULUS OF ASPHALTIC MIXES ACCORDING TO BONNAURE'S NOMOGRAPH (After Bonnaure et al,1977)

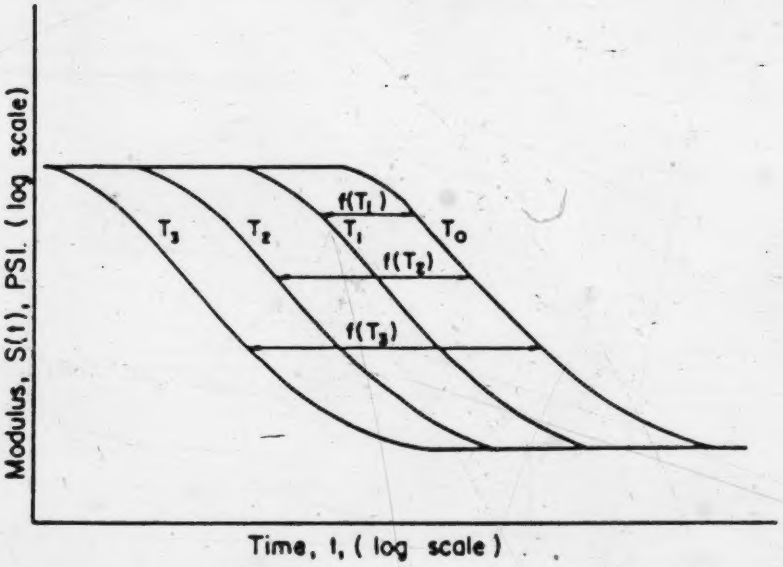


FIG. 2.8 TIME AND TEMPERATURE BEHAVIOUR OF A THERMORHEOLOGICALLY SIMPLE MATERIAL (After Monismith et al,1965)

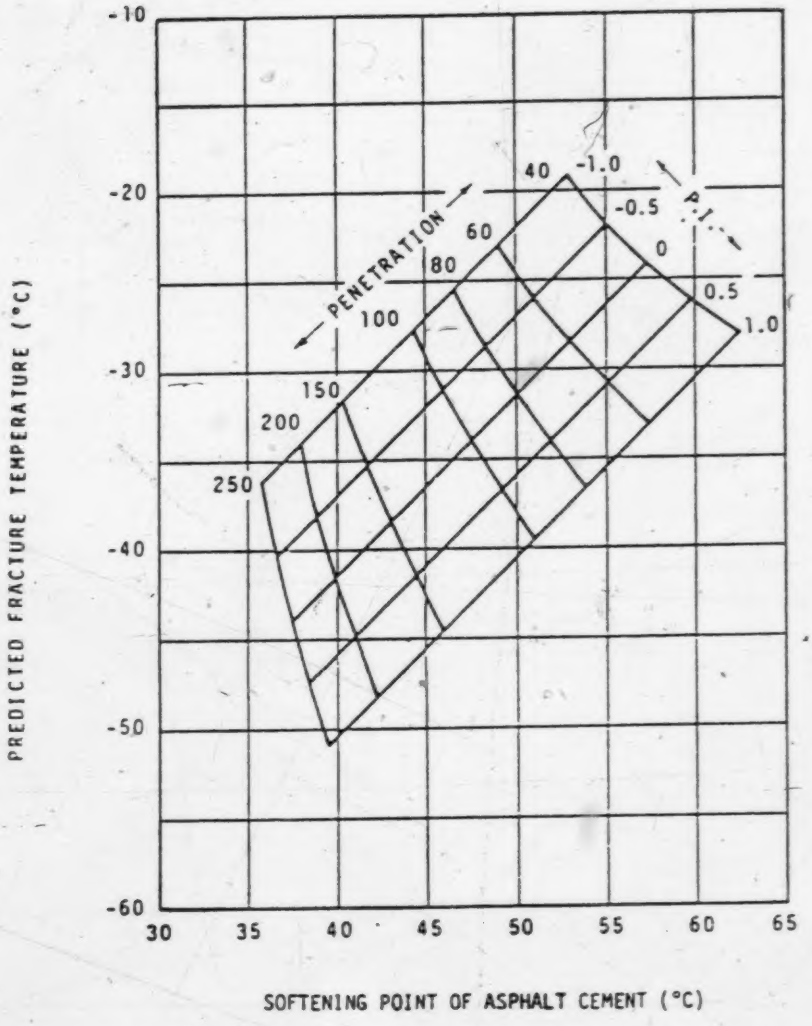


FIG. 2.9 NOMOGRAPH FOR THE PREDICTION OF THERMAL FRACTURE TEMPERATURE (After Sugawara et al.,1982)

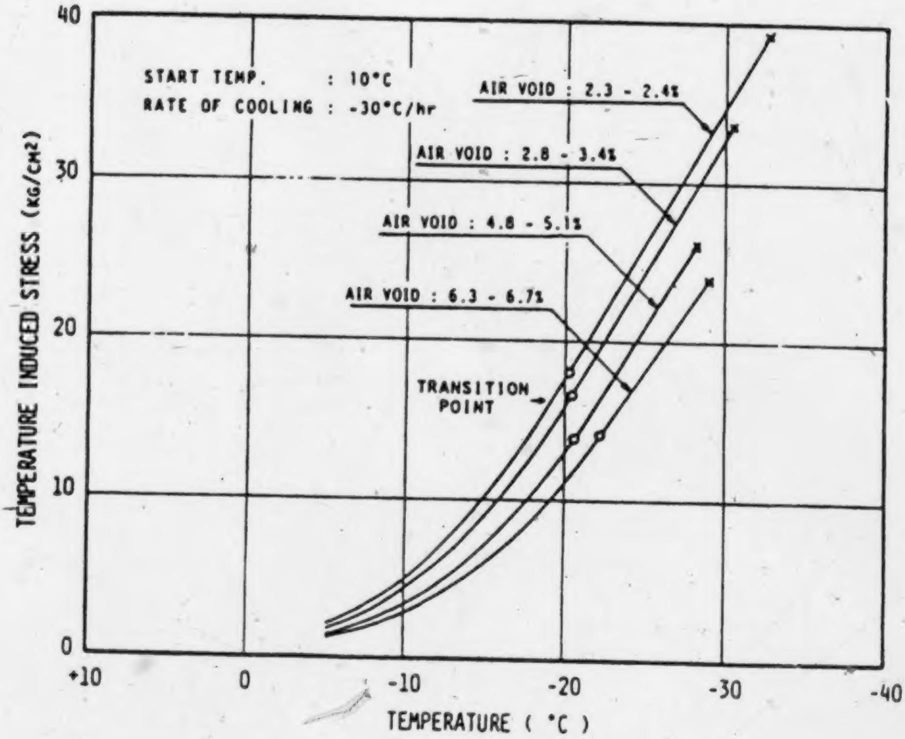


FIG. 2.10 EFFECT OF AIR VOIDS TO THE TEMPERATURE INDUCED STRESS

(After Sugawara and Moriyoshi,1984)

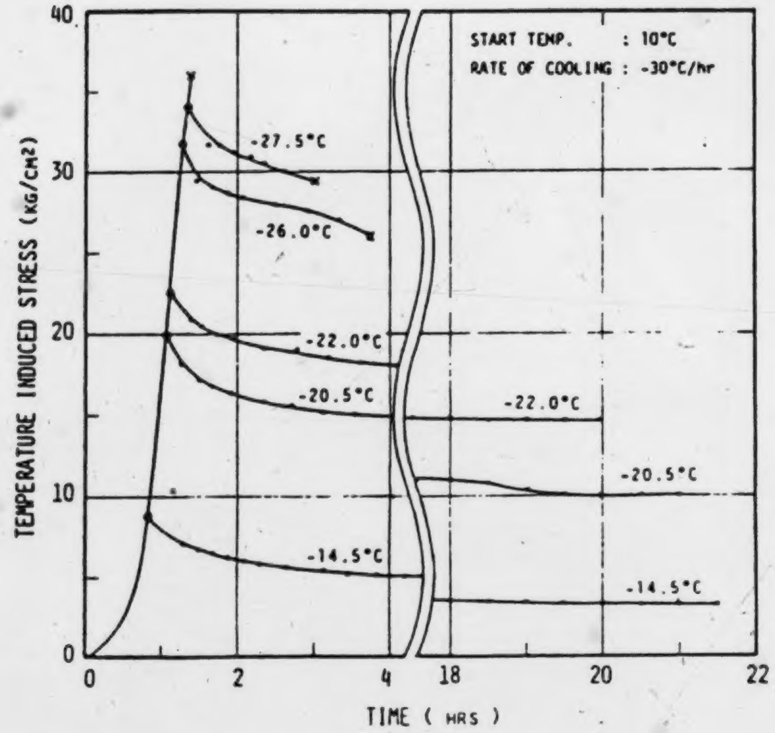


FIG. 2.11 EFFECT OF STRESS RELAXATION TO THE TEMPERATURE INDUCED STRESS (After Sugawara and Moriyoshi,1984)

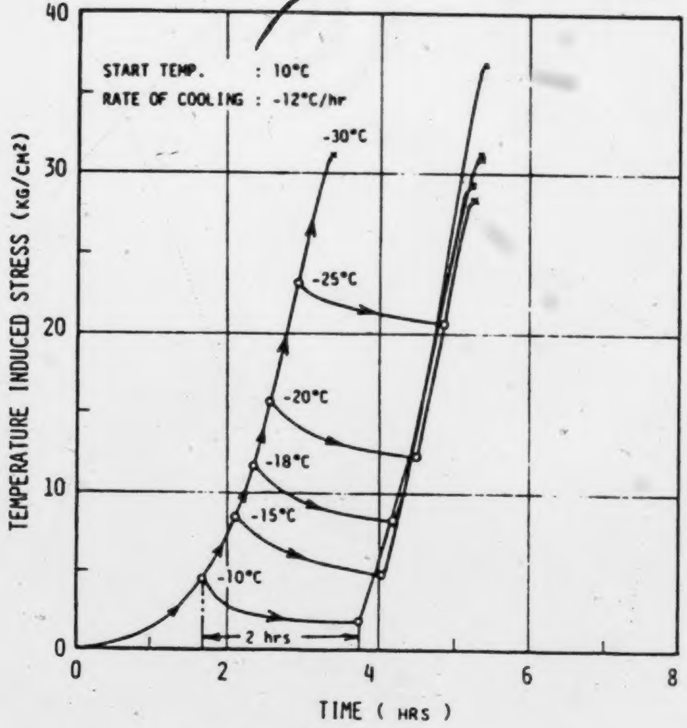


FIG. 2.12 EFFECT OF TWO STAGES COOLING TO THE TEMPERATURE INDUCED STRESS (After Sugawara and Moriyoshi,1984)

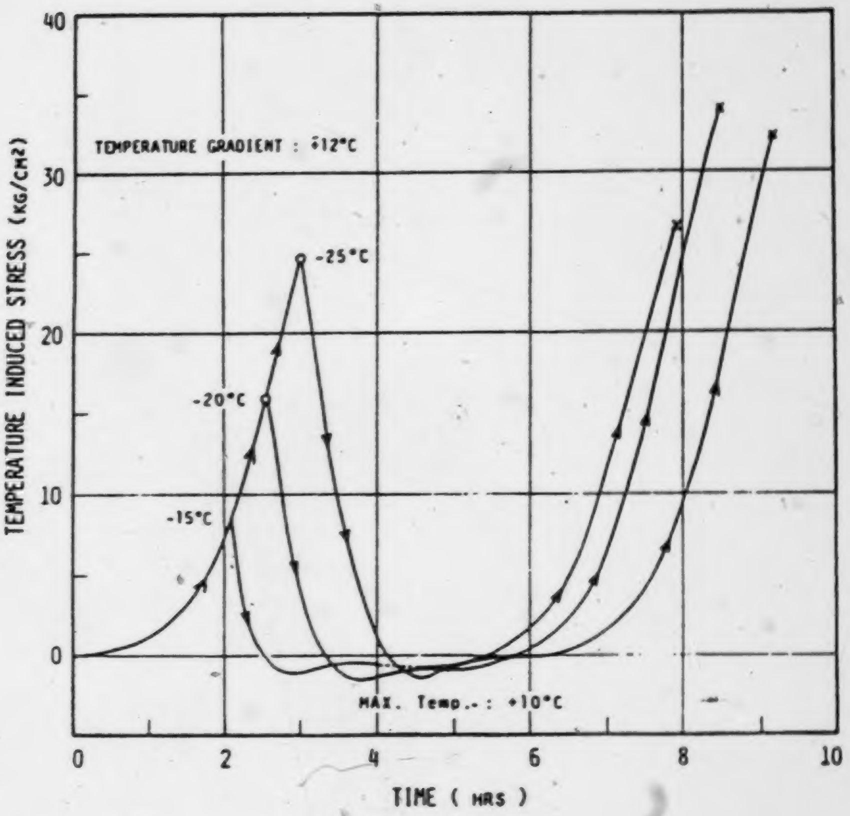
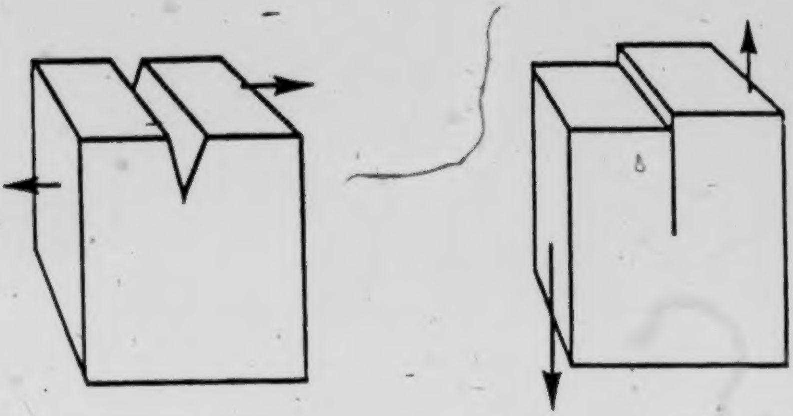


FIG. 2.13 EFFECT OF CYCLIC WARMING AND COOLING TO THE TEMPERATURE INDUCED STRESS (After Sugawara and Moriyoshi,1984)

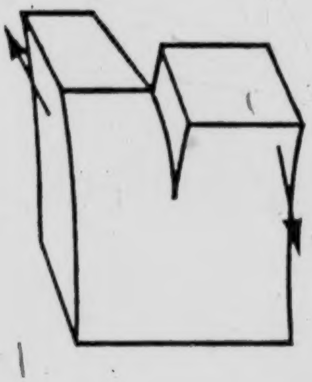


Type I

Tension Opening Mechanism

Type II

Shearing Opening Mechanism



Type III

Tearing Opening Mechanism

FIG. 2.14 CRACK OPENING MECHANISMS (After Langlois,1984)

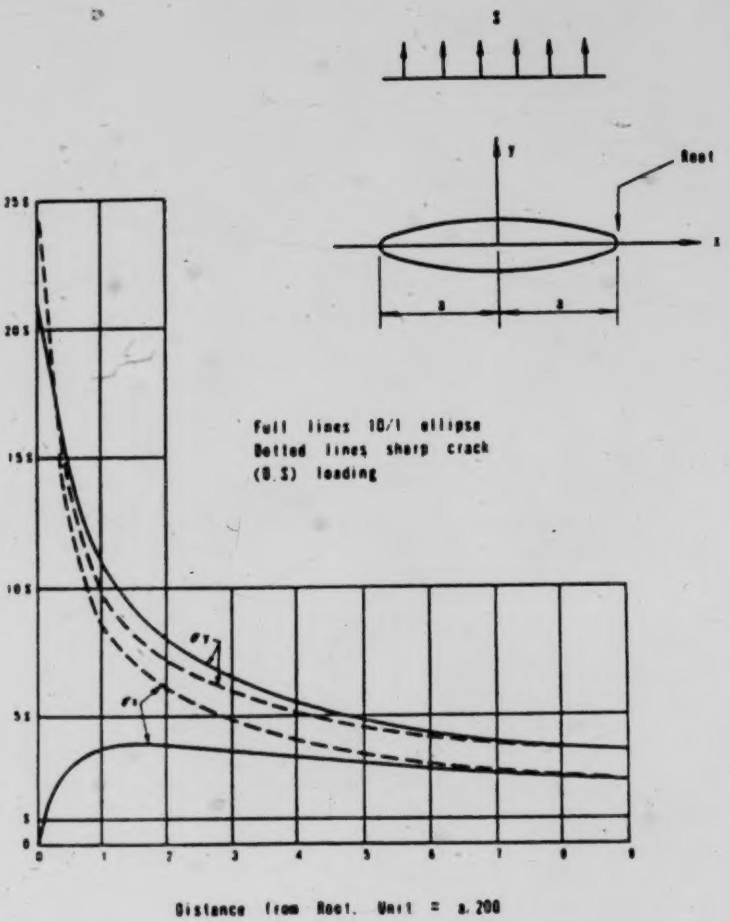


FIG. 2.15 STRESSES IN A CRACKED PLATE UNDER A UNIAXIAL TENSILE STRESS (After Monismith et al,1980)

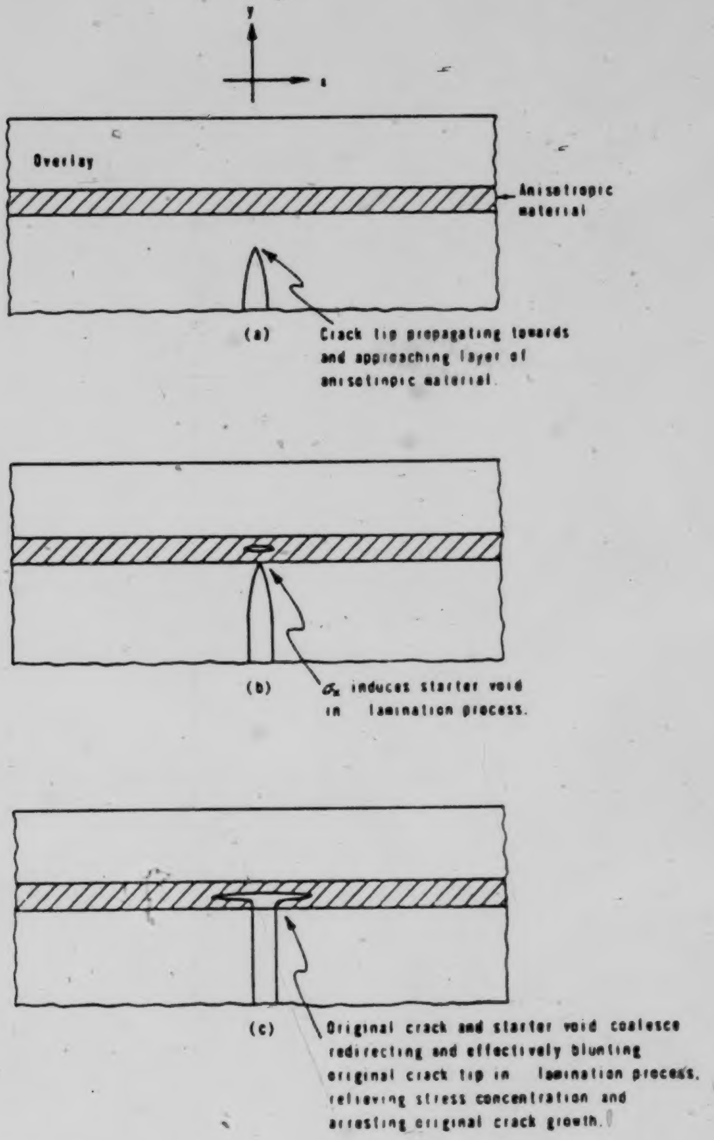

y
x
Overlay
Anisotropic material
(a)
Crack tip propagating towards and approaching layer of anisotropic material.
(b)
σ_x induces starter void in lamination process.
(c)
Original crack and starter void coalesce redirecting and effectively blunting original crack tip in lamination process, relieving stress concentration and arresting original crack growth.

FIG. 2.16 POSSIBLE DELAMINATION PROCESS (After Monismith et al,1980)

CHAPTER 3

PAST EXPERIENCE WITH DIFFERENT REHABILITATION METHODS

3.1 Introduction

As pavement surfaces reach their design life or when their serviceability deteriorates to a terminal level, the need for dependable rehabilitation techniques becomes increasingly important. Rehabilitation procedures include preventing the cracks from existing pavement from reflecting through the overlay, thus decreasing the deterioration rate and prolonging the service life of the overlay.

A comprehensive review of different rehabilitation methods have been presented in the papers by Huffman(1978) and Sherman(1982) respectively. Subsequent information about investigations pursuing this problem is available in different literatures. In this chapter, valuable information of current rehabilitation experience is assembled and reviewed. The underlying theory of each method is briefly discussed, and variations within each method's field performance are also reported.

3.2 Factors Affecting the Performance of Rehabilitation Methods

Sherman(1982) reported that the factors that affect the results of field experiments are:

1) Variability of roadbed performance

Even a sufficiently uniform pavement can have a large variety in performance in different location of the road. Therefore tests should not be done on small, single sections, but rather be done under various conditions and with replicate sections.

2) Difference in climatic condition

Climate is a significant factor to the performance of the pavement. Maximum and minimum temperatures, rainfall, and snowfall in one place can vary widely from year to year. Langlois(1984), in comparing the practice of conventional overlay, scarification, fabric (Petromat) in Quebec with that in other areas, concluded that climate is the factor that reduces the effectiveness of these procedures in preventing reflection cracking.

3) Difference in roadbed preparation

For better evaluation of the effectiveness of a treatment, experiments should be carried out on a uniform pavement. Road preparations before treatment, therefore, is necessary.

4) Presence of construction problems

The results of a field test can be clouded by the presence of construction problems. For field tests to be comparable, it is essential that construction practices be as uniform as possible for all segments of the test.

3.3 Overlay with a Thicker Layer

Stresses induced in overlays in cold areas are mainly caused by temperature differentials and relative movements of the underlying pavement. Thicker overlays help reduce the amount of reflection cracking by increasing the load transfer capability over the cracks. These result in a longer life of an overlay.

Sherman (1982) estimated the thickness of overlay required to retard reflection cracking depending on the following factors:

a) type of pavement being overlaid ---- asphaltic concrete (AC) or Portland cement concrete (PCC);
b) type of distress of the pavement ---- alligator cracks, block cracks, transverse thermal cracks, longitudinal cracks, or PCC joint cracks;
c) climate;
d) number and weight of axle loads.

Langlois (1984) reported the experiments conducted on Quebec highways. Different factors in rehabilitation were investigated for two chosen highway sections. In one route, three different factors, namely:

a) different thickness of layer,
b) addition of asbestos in the mix, and
c) reinforced layer with a tensile membrane,

were tested in seven experimental sections. Though the program was impeded by construction problems, all experimental sections showed little effect on resisting reflection cracking.

Heater scarification with and without the rejuvenating agent Reclamite, together with open graded asphaltic concrete base were tested on another route. Results showed that heater scarification was ineffective for reduction of reflection cracking in cold winter climates such as in Quebec. However, the open graded sandwich layer was very effective when mix application rate was greater than 112.5 Kg/m^2 (approximately 50 mm thick), and the thicker the open graded mix, the less the reflection cracking.

In comparison between the performance of the control section and the conventional overlay tested in Arizona, the time for the crack to reflect to the surface is considerable less in Quebec than in Arizona. A more severe climate in Quebec is suggested to be the reason for this shorter time.

In comparison between the application rate of open mix and conventional mix, Quebec experience concluded that the application rate was the factor that affected the crack resistence. As Fig. 3.1 shows, there is a good correlation between the number of transverse reflection cracks per unit length after 7 years service and the application rate of the paving mix regardless of whether it is a dense mix or an open mix. This relation provides a guide in estimating the required thickness of the overlay.

3.4 Modifying Existing Pavement Surface

Besides the ordinary roadbed preparations, the following techniques, which help prevent the reflection cracking, may also be included in the rehabilitation.

3.4.1 Band-Aid Crack Patching Before Overlay

As reported by Iijima and Matsuno in Miniworkshop of Paving in Cold Areas (PICA) in 1982, experiments using bituminous fabric sheet to patch the crack before overlaying were carried out in Japan. This method has shown a certain degree of success in preventing reflection cracks from coming through in PCC pavement overlay. However when applying this method to asphalt pavement overlay, the effect was unsatisfactory and further investigation was required.

3.4.2 Heater Scarification

Many of the cracks formed in AC pavements are usually Y-shaped with a wide opening at the surface and narrow sharply down to the bottom. By heater scarifying to a depth of approximately 20 mm, the upper portion of the Y-shaped crack together with the sealant are removed. The lower portion of the crack is sealed up by heating. The remixed and recompacted layer serves as an uniform uncracked layer above the crack tip. As a consequence, the reflection cracking of the overlay is slowed down.

A significant amount of work in this area was done, especially in Arizona, Colorado, and Nevada of U.S.A. In

Arizona, heater scarification with Reclamite plus 32 mm wearing course was ranked as the third best among the eighteen test treatments (Way,1980). The percentage of reflection cracking for this project after 6 years was 7.4 percent. However, in Colorado (Donnelly et al.,1976), scarification with Reclamite plus 50 mm wearing course resulted in 100 percent reflection cracking after 5 years. In New Mexico (McKeen et al.,1984), 19 mm scarification with 'rejuvenating agent' plus 16 mm seal coat and 50 mm surface course resulted in 70 percent reflection cracking after 4 years. In Quebec (Langlois,1984), scarification with Reclamite plus 32 mm wearing course had 100 percent reflection cracking after only 2 years. All these suggested that local pavement condition and climate are factors affecting the performance of scarification.

Heater scarification was used prior to overlay on an airfield in Fort Smith, N.W.T. in 1982. Cores taken through a crack before scarification showed that the crack was sealed approximately two thirds of the way through the asphaltic concrete. After scarification, the same crack was again cored, and it was found to be sealed through to the base gravel. There was no significant difference in the amount of reflection cracking in the overlays on scarified or non-scarified pavements after the first winter (Anderson et al.,1984a).

3.5 Stress Relieving Interlayer

The principle of this treatment is to reduce the overlay induced stress by providing an interlayer structure that can absorb small pavement movement, or by providing bond breaking at the interface and thus dissipating stresses at points of crack propagation. Experience in interlayer structures are described in the following sections.

3.5.1 Open Graded Hot Mix (Modified Mix)

The use of open graded hot mix as a stress relieving layer is successful in controlling the reflection cracking (Hensley, 1980). Large percentage of air voids resulted from the open grade helps dampen the movement of the underlying slab and hence reduces the stresses created in the upper binder and wearing courses. The size of the largest aggregate chosen is dependent on the type of pavement to be overlaid. Larger aggregates provide larger air voids which offer more protection against reflection cracking from both horizontal strain and vertical strain mode, the aggregate interlock also gives a better load transfer. Usually larger aggregates are used in PCC pavement overlay where slabs are relatively long, contraction movements expected to be quite large, and deflection at joints is great. Smaller aggregate sizes are used for short slabs or for asphaltic concrete pavements.

Hensley (1980) has reported the use of open graded asphaltic concrete in Tennessee and in Arkansas. This method has yielded excellent results through years. Three gradations of open graded mix have been used successfully. The selection of the gradation of an open graded mix is based on the load transfer capabilities of the cracked areas which are to be overlaid. Regardless of the type grading selected, a minimum of 89 mm open graded course plus a 50 mm intermediate course plus a 25 mm surface course are recommended in Arkansas practice. Over 200 two-lane miles of open graded overlay have been built in the State of Arkansas, and the experience has proved this type of overlay is a viable method for reducing reflection cracking in both PCC and flexible pavement.

McMaster and Blum (1982) reported the experience of open graded mix by the Ontario Ministry of Transportation and Communication (MTC). The mixes they employed were types H.L.4 and H.L.8 (Table 3.1) and were used on AC pavements and on PCC pavements respectively. It has been concluded that the most effective technique employed by the MTC in its attempt to control reflection cracking has been the use of modified hot mixes as lower binder courses.

Quebec's experience on open graded mix was reported by Langlois(1984). Compared to the Arkansas experience, the open graded mixes employed in Quebec are finer. Moreover, for economy, the intermediate layer was eliminated. The results show that open graded mix is effective in reducing reflection cracking. However, as reported in Section 3.3, Langlois concluded that the application rate is more important than the gradation of the mix for reducing reflection cracking. A comparison of the gradation of open graded mixes used in Arkansas, Ontario and Quebec is shown in Table ~~3.1.~~

3.5.2 Geotextile (Fabric)

The function of fabric in this method is not to reinforce, but to act as a stress relieving layer. It allows small movements of the underlying slab without inducing high stresses to the overlay. The types of fabric most commonly used include fibreglass, nylon, polyester and polypropylene. Typical properties of these material compared to the concretes (AC and PCC) are shown in Table 3.2.

Yuce et al.(1983) used finite element analyses to study the stress effect of fabric on asphaltic concrete overlays on PCC pavements. In the study, a static uniformly distributed load equivalent to a 18,000 axle load was simulated. Temperature induced stress was also considered in the analyses. A crack, which extended through the pavement structure, was treated as a low modulus element with the same coefficient of thermal expansion as the fabric. The properties of various fabrics are shown in Table 3.3.

The analyses showed that the tensile stress and strain at points of crack tips on the underside of the asphaltic concrete overlay were lowest when the fabric was about 25 mm above the crack pavement. Since reflection cracking could be mitigated by reducing stresses at the crack tips, placing fabric at this position would have the best results in reflection cracking control. The analyses also found that thicker fabric is more effective in decreasing stress at the crack tips and thus is desirable in resisting reflection cracking.

The field performance of fabric (Petromat) on AC pavements in Colorado, with a freezing index about 1500 C-days, was reported to be very successful with only 2 percent reflection cracking in the 64 mm wearing course after 5 years. However, in Quebec, freezing index about 2500 C-days, Petromat did not perform so well (Langlois,1984). After 4 years, 47 percent of the cracks reflected through the 50 mm wearing course.

The performance of fabric under an extremely cold climate at Thule Airbase in Greenland was reported by Eaton and Godfrey (1980). The freezing index there was about 4600 C-days. Different weights of Petromat and Bidim were used. The results showed that where the AC 2.5 overlay thickness was less than 50 mm, 71 percent of the cracks reflected through the first year and 105 percent the second year. For the overlay greater than 75 mm, 46 percent reflected in the first year and 57 percent the second year. Finally the authors concluded that although fabric type or weight did not significantly influence the rate of reflection cracking, the fabrics do serve as a waterproofing membrane.

Fabrics were also tested on two test roads in New Mexico, freezing index 0 to 150 C-days (McKeen et al.,1984).

Two non-woven fabrics, one consisting of polypropylene and nylon and another of needle punched polypropylene, were tested on both of the two test roads. Different tack rates were applied to these fabrics. On one test road, the tack rate to these fabrics were 0.92 and 1.47 l/m^2 respectively. On another test road, the tack rate were 0.82 and 1.36 l/m^2 respectively. After about 4 years, the percentage of cracks reflected to the surface on the first road were 100 and 80 percent respectively. On the second road, after about 5 years, the reflection rate were 30 and 22 percent respectively.

The performance of fabric is dependent on the successfulness of its installation. A typical construction sequence was described by Dykes (1980). The amount of tack coat applied to the fabric is also an important factor. Dykes recommended that the rate applied must be sufficient to seal the fabric and provide bond to adjoining pavement layers, yet not "flush" to the newly paved surface.

Smith (1983), by relating the amount of tack coat (liquified asphalt) to the weight and thickness of the fabric, suggested the following regression equation:

$$RTC = 0.055\ (T\ W)^{0.3} \tag{3.1}$$

where RTC = recommended tack coat rate (gal/yd^2)

T = fabric thickness (mils)

W = fabric weight (oz/yd^2)

Sherman (1982) suggested the rate of asphalt applied depended greatly on the pavement condition. When applied to tightly sealed surfaces, or on steep grades, the rate might be reduced about 20 percent. When applied to highly porous or ravelled surfaces, the rate might be increased about 20 percent. He also found that penetration grade asphalt provided a better bonding between fabric and pavement than emulsified asphalt. McLaughlin (1979) also suggested a correction of application rate based on the existing pavement surface condition (Table 3.4). The grade of the asphalt cement used for tack coat should depend on the pavement surface temperature (Fig.3.2).

3.5.3 Soft Asphalt Interlayer

A softer grade asphalt can substantially alter the elastic modulus of the asphaltic concrete thus reducing the crack tip stresses.

The use of soft asphalt interlayer has been successful in a series of experimental projects. In 1971, an experimental project in Wyoming showed that a 50 mm soft asphalt interlayer (AC 2.5F) and crack sealer produced the least cracks and was the most effective for reducing reflection cracks (Sherman,1982).

In Arizona (Way,1980), the 200-300 penetration asphalt from the Los Angeles Basin (low temperature susceptibility) added to a 32 mm asphaltic concrete overlay and then covered with a 13 mm asphaltic concrete finishing course was found to be one of the five most effective treatments to reduce reflection cracking. This overlay structure complied with the recommendations of the report by Carpenter et al.(1976)

> The best overlay design to reduce the appearance of cracking is:
> a) a thin layer with soft asphalt (low η) and low modulus of elasticity to serve as a stress relieving medium overlaid by,
> b) a layer with soft asphalt (low η) and a high modulus of elasticity.
> Although this arrangement will hasten the propagation of unseen cracks through the surface of old pavement, it will slow them down considerably when they reach the surface and contact the underside of the stress-relieving layer.

3.5.4 Asphalt Rubber Interlayer

Asphalt rubber is made from mixing relatively high concentrations of reclaimed rubber in hot asphalt. When comparing the amount of reflection cracking alone, asphalt rubber membrane covered by a 13 mm AC finish course was the best among the eighteen treatments tested in Arizona (Way,1980). However an uncomfortable amount of shoving can occur which leads to an undesirably rough ride.

Asphalt rubber interlayer is also known as Stress Absorbing Membrane Interlayer (SAMI). Vallerga et al.(1980) reported on the proper construction practice of SAMI and a few successful uses of SAMI in Arizona projects. New Mexico (McKeen et al.,1984) used factorial design to study the influence of variables such as rubber type, mixing temperature, storage period and storage condition, mixing time, batch repetition and test temperature. Four laboratory experiments and one field trial were conducted. Results from the field experiment showed that the mixing time has a significant influence on cracking observed while the rubber type showed no influence on cracking.

Morris et al.(1982) used finite element analyses to study the effect of the inclusion of an asphalt rubber layer in a cement treated base AC overlay structure. Results of this study indicated that the horizontal tensile stresses near the crack tips could be reduced significantly as a result of the SAMI. Overlay thickness in a SAMI structure has only slight influence on the stresses near the crack tips.

3.6 Asphalt Overlay Material Modification

As discussed in Chapter 2, both the tensile strength and the induced stresses are proportional to the stiffness of the mix which in turn is affected by the stiffness of the asphalt cement. Therefore the grade of an asphalt, as well as its temperature susceptibility, are important factors for the cracking resistibility of the pavement. Specifications for choosing the right type of asphalt for the mix and techniques to improve the rheology of asphalt are discussed in the following sections.

3.6.1 Soft Grade Asphalt

One of the reasons for the cracking of a bituminous pavement is its brittleness at low temperature when the asphalt it contains becomes hard. In the paper concerning the asphalt specifications in British Columbia for low temperature performance, Read[illegible]1972) reached a conclusion that in regions of very low service temperature (below -40 C), bitumen used in British Columbia should be at very soft consistencies if cracking was to be avoided.

Concerning the use of soft asphalt, Huffman (1978) made the following comment:

> From this and other studies, it may be surmised that asphalt stiffness can play a significant role in the amount of reflection cracking. However, eventually after sufficient asphalt aging, significant reflection cracking will occur. When contemplating the use of soft asphalts for overlays, the potential problems of mix bleeding and rutting must also be considered.

3.6.2 Modified Asphalt

Ideally, it is best to choose an asphalt whose temperature susceptible characteristics satisfies both ends of the temperature performance scale, an asphalt that is stiff at high summer pavement temperature and soft at low winter pavement temperature. Some of the attempts to modify the susceptibility and consistency of asphalts are discussed in the following sections.

3.6.2.1 Air Blowing

Some of the asphalts processed in Western Canada are waxy and have low viscosity. The wax content in these asphalts makes the asphalts temperature susceptible and results in excessive cracking at low temperatures. Chemists and asphalt technologists have tried to improve these waxy asphalts by reducing their temperature susceptibility and hence getting a better mix stiffness at low temperature.

Air blowing (i.e. oxidation) is one process proved to be successful in improving waxy asphalt temperature susceptibility. Experience showed that air blowing could increase the PI of an asphalt, i.e. decrease the temperature susceptibility of an asphalt. If one accepts the critical stiffness theory, this will allow use of harder grade asphalt cements. The strength and fatigue resistance of the pavement will be increased and the potential for plastic deformation will be decreased. The extent of air blowing required to produce the desired PI is still not thoroughly known.

Gaw et al.(1976) reported the experience of air blown asphalts in two test roads. The first test road is near Richer, Manitoba. The asphalts used were BLV 100-150 and BLV 150-200. After two years of service, the blown asphalts showed significant improvement in crack resistance when compared with the same grade of asphalt used in the Ste. Anne Test Road.

Another test road is in northern Ontario. Blown LV 150-200 and LV 85-100 asphalts were used in overlaying in this project. In order to isolate the performance of blown asphalts from the influence of underlying pavement, a layer of 0.3 m granular material was put over the cracked pavement first before overlaying. After three years of service with minimum temperature reaching -38C, the blown asphalt sections showed no significant transverse cracking and their performance were better than that of the HV 150-200 asphalt which served as the control.

Saskatchewan experience in air blown asphalts was reported by Clark and Culley(1976) and Culley(1983). The air blown asphalts used had an average penetration of 88 and 139 and were designated as 100AB and 150AB respectively. After five years, the rheology changes of the blown asphalts were comparatively less than a better grade flashed asphalt, the 200 pen AC 5 asphalt. It was concluded that the resistance to aging of the air blown asphalt would result in fewer cracks in the long term.

3.6.2.2 Sulfur Asphalt

Asphalt rheology can also be improved by means of additives. Sulfur has been tested as a beneficial additive to asphalt. Besides its success in controlling reflection cracking, replacing high value asphalt by low price sulfur will also result in a relatively lower cost of asphaltic mix.

Hignell et al.(1972) described the sulfur asphaltic concrete as an ideal mix, a mix which is optimum for both low temperature and high temperature conditions. Meyer et al.(1977) said that the addition of sulfur to the asphalt cement would not alter the low temperature property of the asphalt. The property was controlled by the grade of the asphalt only. However, the addition of sulfur could increase the stability and stiffness of the asphaltic concrete at high temperature. Thus sulfur asphalts can have the advantage of soft grade asphalts at low temperatures to prevent the cracking problem but does not have the rutting problem that happens in soft grade asphalts at high temperatures. The stiffness of a sulfur asphalt at higher temperatures is determined by the amount of sulfur added.

In the years of 1975 to 1979, as reported by Fromm and Kennepohl (1979) and Fromm et al.(1981), four test roads using sulfur asphaltic concrete mixes had been constructed. Sulfur/asphalt weight ratios of 50/50, 40/60 were used in different test sections.

Sulfur fumes at paving operation were encountered in these projects. The fumes were identified as hydrogen sulfur and sulfur dioxide, though their concentrations were far below toxic level. The amount of fumes produced was directly related to the mix temperature. It was found that if mix temperature was kept below 130 C, the fumes would be considerably less. While careful control of mix temperature can significantly reduce the fumes, sulfur fumes will still occasionally occur especially during warm and humid days.

In 1981, the performance of the test sections indicated that pavements paved with sulfur asphalt were better in resisting thermal cracking than pavements paved with the regular grade of asphalt cement. Sulfur also increased the stability of an asphaltic concrete to resist rutting and deformation of the pavement. The report showed that when the rut depths of the test sections were compared to the control sections, sulfur asphalt of a softer grade had a smaller rut depth than the control sections which were paved with stiffer grade asphalts.

Kandhal (1982) reported the laboratory tests and the field experiements of sulfur mixes. Six levels of sulfur/asphalt weight ratios, 0/100, 10/90, 20/80, 30/70, 40/60 and 50/50, were tested in the laboratory to study the change of mix properties with respect to temperature. The tests showed that as the temperature was lowered from 25 C to -29 C, the tensile strength of the mixes increased regardless of the sulfur content. Between 25 C and -12 C, there is no significant difference between the tensile strength of various mixes. At lower temperatures, -23 C to -29 C, the tensile strength of the mix was affected by the sulfur content. As for the stiffness moduli of the mixes, the sulfur content is less significant at low temperature. At higher temperatures, the effect of sulfur becomes more prominent. The mix having a 50/50 sulfur content had the highest stiffness value through the entire test temperature range. Three experimental sections were overlaid with 30/70 sulfur asphaltic concrete in 1980 in Pennsylvania. No significant change was oberved between the performance of the test sections and the control section up to the time they are reported.

3.6.2.3 **Rubber Asphalt**

Rubber particles when mixed with asphalt cement at 190C swell to about twice their original volume. In addition to swelling, the rubber particles become softer and more elastic. This change gives additional "stretchability" to the mix, enabling it to withstand larger strain without breaking. Although this change in properties can reduce the cracking tendency, it is detrimental to stability.

The Arizona experience of various rubber asphalt overlay structures in reducing reflection cracking was reported by Vallerga et al.(1980). The rubber asphalt contained 20 to 25 percent of rubber. The overlay structures included:

a) Stress Absorbing Membrane (SAM), a chip seal coat like surface layer structure with thickness ranged from 9 to 19 mm;

b) Stress Absorbing Membrane Interlayer (SAMI), a sandwich layer structure with asphalt rubber mix as the middle layer; and

c) Plant-mixed SAM, a structure similar to conventional mix in which rubber asphalt was mixed with proportional aggregates in a conventional mixing plant and the mixture was then placed directly on grade.

All these structures performed well in preventing reflection cracking. Little or none of the transverse cracks reflected to the surface in their first four to five years of service. In cases where cracks did reflect to the surface, they remained narrow and did not spall; no maintenance was required. However, tenderness of the overlay, especially in extremely hot weather in summer, caused problems in some areas.

The Connecticut experience was reported by Stephens (1982). Mixes were modified by adding 1 or 2 percent of rubber. Nine test sections were used to study the performance of rubber modified asphaltic concretes in different pavement distress conditions and in different traffic levels. The results showed that sections overlaid with one percent rubber modified mix, on an average, had fewer cracks than the nonrubberized section. However, sections overlaid with two percent rubber modified mix had more cracks than the non-rubberized section.

3.6.2.4 Other Additives

Other additives such as asbestos fibers, carbon-black and other metal additives are also found to be successful in modifying asphalt. Asbestos fortified AC mix was rated as one of the five treatments that significantly reduced reflection cracking in Arizona (Way,1980). However as asbestos has been identified as a carcinogenic substance, asbestos in asphalt overlays is no longer used.

The use of carbon-black to modify asphalt had been studied by Rostler et al.(1977). It was found that addition of this microfiller could decrease the temperature susceptibility of the asphalt and retard the hardening of the mix during hot plant mixing. Subsequent research has been reported by Vallerga and Gridley (1980) on the beneficial effect of carbon black on various properties of asphaltic concrete. Recent laboratory studies on the behaviour of asphaltic mixtures with carbon black reinforcement have been presented by Yao and Monismith (1986).

Asphalt can also be modified by adding a small amount of metal that triggers polymerization of the asphalt (Kennedy et al.,1981). Laboratory results showed that the temperature susceptibility of the asphalt was reduced by this modification. Indirect tensile strength and bending strength of the modified mix were reported to be increased. Further study of metal additive using manganese as the modifier was reported by Kennedy et al.(1985). The findings indicated that the manganese increases the high temperature stiffness, strength, and stability and decreases the low temperature stiffness of asphaltic mixtures. However the tensile strength of the modified mix at low temperatures also appears to be lower than the conventional mix.

3.7 Pavement Reinforcement

Construction techniques using reinforcing elements of high tensile strength to strengthen the tension ability of another material have been applied in many occasions. The use of steel reinforcing bars in reinforced concrete is a good example of such construction techniques. Applying the same concept, reinforcing elements, both metallic or nonmetallic, have been considered in asphaltic concrete.

Welded wire reinforcement to bituminous concrete has been tried since the fifties. Even now, wire mesh is still being used in some experimental projects as a reinforcement to asphaltic concrete. Kubo et al.(1984) reported the use of a crimped wire mesh in Minami-Furano, Japan. There the effect of the wire mesh was inconclusive since many problems had been created during construction. Some of the problems in using wire mesh were reported by Davis (1960).

A high strength plastic geogrid known as TENSAR was introduced in late 1980 and was reported by Abdelhalim et al.(1982) and Kennepohl et al.(1985). This material is made from polypropylene and is biaxially oriented to give strengths in the order of mild steel in both directions. Material testing research of TENSAR was carried out first at Royal Military College (RMC), then at the University of Waterloo.

The research results showed that TENSAR was effective as a reinforcement of bituminous layer, of base and of subgrade. The tests evaluated the fatigue strength of the asphaltic concrete structure. The results showed that a 150 mm reinforced asphaltic concrete layer could carry as many cycles of load as a 250 mm unreinforced asphaltic concrete layer.

Computer analysis using 'Texas Transportation Institute Overlay Tester', a program simulating crack growth due to thermal contraction, indicated that the presence of the reinforcing grid improved the fracture retarding performance of the mix (Kennepohl, 1984). Also, testing at the RMC showed that at the end of the loading cycles, the test sections reinforced with TENSAR were still relatively sound while the non-reinforced sections were seriously cracked. (Abdelhalim e.al.,1982). These findings gave a positive indication to the crack resisting ability of the TENSAR.

Field trials are necessary to determine if the laboratory evidence can be related to field performance. A number of field trials using TENSAR as a reinforcement for asphaltic concrete was designed and constructed in 1984 in Canada, the United States and Europe. Results from these trials will be of great interest.

Another reinforcing element has been recently developed. It is made of fibreglass and is called GLASGRID (R. Shoesmith and J. Emery, 1985). This material is promising because fibreglass is a relatively cheap material having a high modulus and low percentage elongation. Table 3.5 shows the properties of GLASGRID in comparison with other fibric meshes.

Flexural tests of GLASGRID reinforced asphalt concrete prisms showed that the reinforced prisms were about two and a half times stronger in flexure than unreinforced prisms. Moreover, cracks did not propagate through the reinforced prism overlay when it failed. Instead, horizontal delamination of the reinforcement from the underneath layer resulted. After fracture, the GLASGRID was observed to be fully bound to the base of the overlay and holding the fragments together. These cracking phenomena were desirable in overlay construction. Field trials of this GLASGRID were carried out in a number of places. Successful installations of this GLASGRID in different demonstration sections were reported. An experimental section has been constructed in Alberta in 1985 incorporating both TENSAR and GLASGRID (McMillan and Gavin,1986).

3.8 Summary of Experience

Different degrees of success have been reported in all the techniques discussed in the above sections. A technique reported to be successful in one place may not be as successful in another area. Differences may be caused by a) variability of roadbed performance, b) difference in climatic condition, c) difference in roadbed preparation, d) presence of construction problems.

Of the different kinds of approaches used to control reflection cracking, i.e. thicker layer, modifying existing pavement surface, stress relieving interlayer, asphalt overlay material modification and pavement reinforcement, it appears that stress relieving interlayer using open graded mix and asphalt overlay using a lower consistency asphalt or modified asphalt perform better in inhibiting the reflection cracking. Initial laboratory testing of reinforcement, such as TENSAR and GLASGRID, appears to be promising. Field trials performance information is needed to evaluate their effectiveness in practical use.

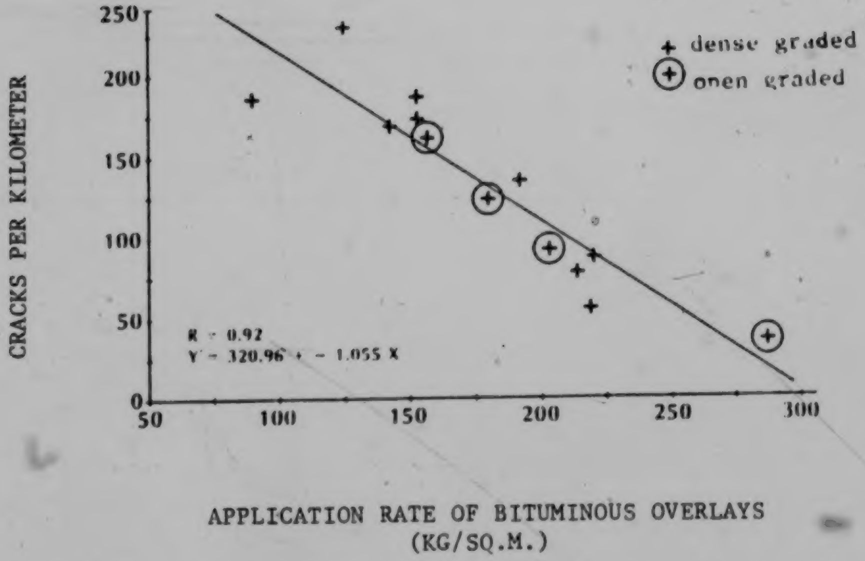


FIG. 3.1 RELATION BETWEEN THE APPLICATION RATE OF MIX AND THE NUMBER OF TRANSVERSAL REFLECTIVE CRACKS (After Langlois,1984)

TABLE 3.1 COMPARISON OF OPEN GRADED MIXES USED IN ONTARIO, ARKANSAS AND QUEBEC

	ONTARIO		ARKANSAS			QUEBEC	
PAVEMENT OVERLAID	H.L.4 AC	H.L.8 PCC	Mix A PCC	Mix B PCC/AC	Mix C AC	Route 175 AC	Route 161 AC
SIEVE DESIGNATION				% PASSING BY MASS			
76 mm	-	-	100	-	-	-	-
64 mm	-	-	95-100	100	-	-	-
51 mm	-	-	-	-	100	100	-
38 mm	-	100	30-70	35-70	75-90	75-90	-
26.5 mm	-	70-100	-	-	-	-	100
19 mm	-	50-100	3-20	5-20	50-70	50-70	96
16 mm	100	43-90	-	-	-	-	-
13 mm	70-86	-	-	-	-	-	54
9.5 mm	30-52	15-55	0-5	-	-	-	32
4.75 mm	5-15	5-15	-	-	8-20	8-20	7
2.35 mm	0-10	0-10	-	0-5	-	-	4
Fine	0-4	0-4	-	0-3	0-5	0-5	3
ASPHALT CONTENT BY MASS	2.5-3.5%		1.5-3.0 %			2.3 %	

TABLE 3.2 TYPICAL PROPERTIES OF FIBER MATERIALS AND CONCRETES

(After Shoesmith and Emery,1985)

Fiber Materials	Tensile Strength GPa	Young's Modulus GPa	Elongation at Break %	Specific Gravity	Specific Strength GPa	Specific Modulus GPa
Asbestos (crocidolite)	3.50	196	2-3	3.37	1.04	58.2
Carbon (high tensile)	2.60	230	~1	1.90	1.37	121.1
Glass (ECR, filament)	2.07	76	~4	2.54	0.81	29.9
Kevlar (PRD 49)	2.90	133	4	1.45	2.00	91.7
Nylon (Type 242)	0.75-0.90	up to 4	13.5	1.14	0.79	3.5
Polyester	1.04	10	-	1.40	0.74	7.1
Polypropylene (filament)	0.40	up to 5	18	0.90	0.44	5.5
Steel (high tensile)	0.70-2.00	200	3.5	7.86	0.25	25.4
Concretes						
Asphaltic concrete						
(0°C)	-	17	-	2.40	-	7.1
(20°C)	-	2.7	-	2.40	-	1.1
(40°C)	-	0.4	-	2.40	-	0.17
Portland cement concrete	0.001-0.004	30-40	0.005-0.015	2.38	0.002	16.9
Portland cement mortar	0.002-0.004	25-35	0.005-0.015	2.25	0.002	15.6

TABLE 3.3 PHYSICAL PROPERTIES OF FABRICS (After Yuce et al.,1983)

Type of Fabric	Weight (oz/yd^2)	Thickness[1] (mils)	Grab Tensile[2] Strength (psi)		Elongation (Percent)[2]		Secant Modulus[3] (psi)	
			Machine	Cross	Machine	Cross	Machine	Cross
Petromat (Phillip Fibers)	4.5	40	81	132	85	74	2,294	1,483
Bidim C-22 (Monsanto)	3.2	51	125	96	90	98	1,878	871
Bidim C-34 (Monsanto)	9.6	77	178	151	57	73	1,939	1,731
True Tex Mg 75 (True Temper)	6.5	56	170	98	96	97	1,936	666
True Tex Mg 100 (True Temper)	6.5	88	174	114	94	116	896	414
Duraglass B-65 (Johns-Manville)	9.8	77	126	116	3	3	tear	tear
Q Trans-50 (Quline)	7.0	105	93	142	173	107	350	160
Fibretex 200 (Crown-Zellerbach)	6.0	73	183	126	145	175	1,025	368
Reepav 376 (Dupont)	3.0	14	110	79	63	64	4,650	3,250
Nicofab B50 (Nicolon)	4.9	68	80	133	100	79	1,339	552
Amopave 4545 (Amoco)	6.6	40	142	147	73	104	1,890	1,480
Trevira 1117 (Hoechst)	4.4	51	162	119	82	111	1,666	810

*All values are from Trans Lab Testing.

[1] ASTM 461

[2] ASTM 1117; 1-in. grip

[3] At 50 percent strain, unless tearing occurs

TABLE 3.4 ASPHALT APPLICATION RATE CORRECTION DUE TO EXISTING PAVEMENT SURFACE CONDITION (After McLaughlin,1979)

Description of Existing Surface	Approximate Surface Texture, cubic inch per square inch (cm^3/cm^2)*	Asphalt Quantity Correction, gallon per square yard (ℓ/m^2)**
Flushed Asphalt Surface	0.001 to 0.005 (0.002 to 0.013)	-0.06 (-0.27)
Smooth, Nonporous Surface	0.005 to 0.015 (0.013 to 0.038)	-0.03 (-0.14)
Slightly Porous, Slightly Oxidized Surface	0.015 to 0.025 (0.038 to 0.064)	0.00
Slightly Porous, Oxidized Surface	0.025 to 0.040 (0.054 to 0.102)	+0.03 (+0.14)
Badly Pocked, Porous, Oxidized Surface	0.040 and above (0.102 and above)	+0.06 (+0.27)

*Putty Method

**Correction to standard tack coat of 0.18 gal/yd^2

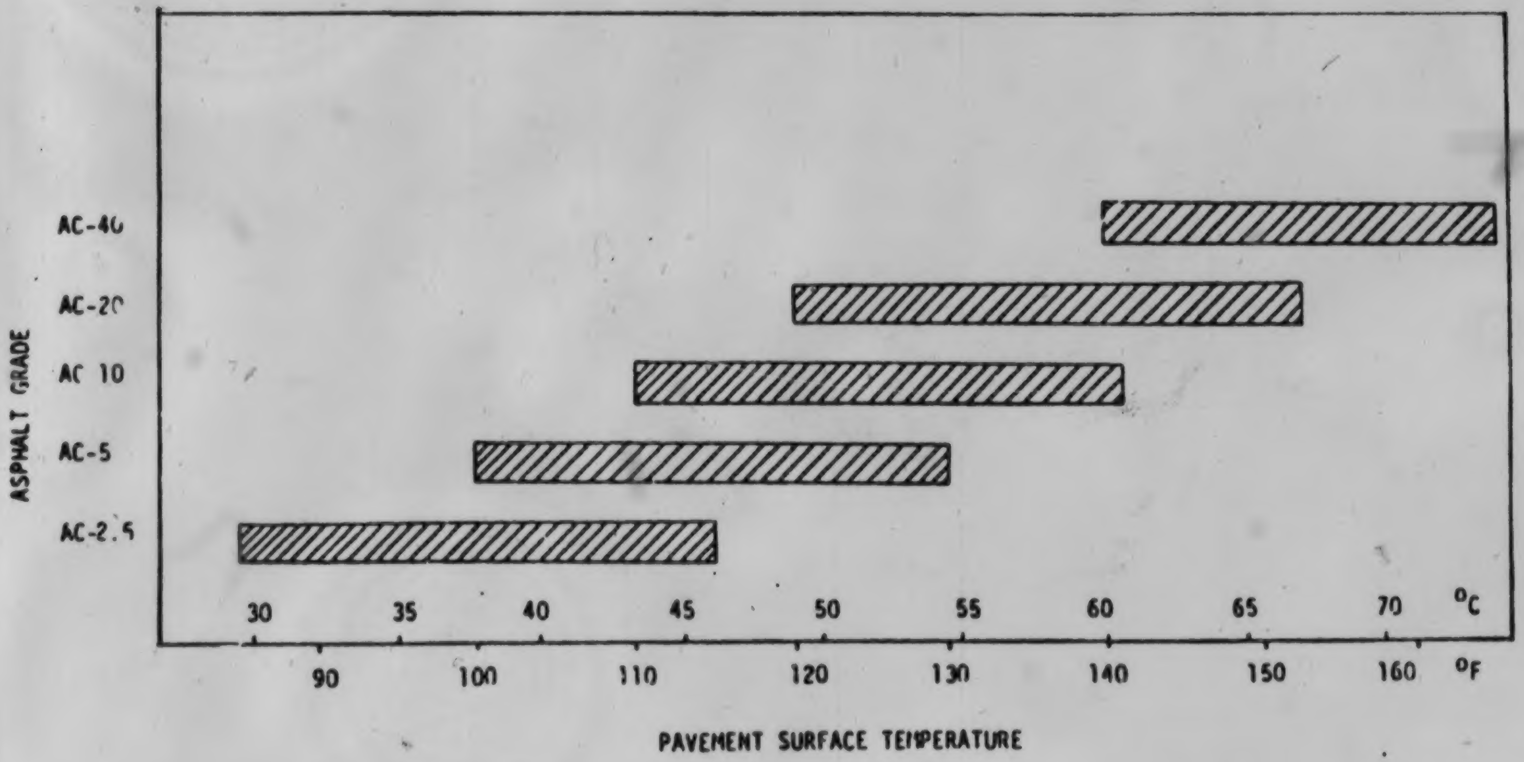


FIG. 3.2 ASPHALT GRADE OF TACK COAT IN ACCORDING TO THE PAVEMENT SURFACE TEMPERATURE
(After McLaughlin,1979)

TABLE 3.5 TYPICAL FABRIC AND MESH PROPERTIES (After Shoesmith and Emery,1985)

Name	Composition and Manufacture	Weight g/m^2	Mesh Aperture mm	Ultimate Strength kN/m	Elongation at Ultimate Stress %
Bidim^R U34	100% polyester, needled	270	fabric	14.0	60-80
Fibertex^R S-300	polypropylene fibers needle-punched and thermic bonded, precompressed	300	fabric	12.0 length 16.0 cross	40-60
Petromat^R	polypropylene needle-punched and sealed with asphalt cement	140	fabric	20.1	65
Typar^R 200	100% polypropylene melt-bonded	200	fabric	16.3	52
Tensar^R SS-2	100% high density polyethylene biaxially orientated	320	37 x 25	36.2/17.0	12
Tensar^R AR-1	100% high density polyethylene biaxially orientated	263	72 x 51	27.4/16.1	~12
Glasgrid^R	glass fiber grid, bitumen coated	300	25 x 12.5	100/50	~4

CHAPTER 4

YELLOWKNIFE OVERLAYING PROJECT

4.1 Background

Many airfields now under the jurisdiction of Transport Canada in the Western Canadian region were built during the war time in support of the two major war time projects, i.e. the Canol Project and the Alaska Highway. The former project includes airfields such as Ft. Smith, Hay River, Yellowknife and Norman Wells; while the latter project includes airfields of Grand Prairie, Ft. St. John, Ft. Nelson, Watson Lake and Whitehorse. These airports are maintained and expanded by the Federal Government in support of the expanding social program and the natural resource development in the north.

In the summer of 1983, Transport Canada carried out a research program prior to and in conjunction with the overlay construction of runway 15-33 at Yellowknife Airport. The construction contract called for the paving of runway 15-33, taxiway A, aprons 1 and 2 and portions of the air terminal building access road. It was tendered with two alternative construction methods:

Alternative A --- levelling course and overlay 75 mm± for runway and 65 mm± for apron and taxiway. Hot mix asphalt placed in two compacted layers of 40 mm maximum plus levelling course as required.

Alternative B --- Heater scarify existing pavement surface.

Levelling course and overlay 65 mm± for runway and 55 mm± for apron and taxiway. Hot mix asphalt placed in two compacted layers of 40 mm maximum plus levelling course as required.

As a result of the bid prices, Alternative A was selected for construction. Peter Kiewit Sons Co. Ltd. was the prime contractor and Associated Engineering Services Ltd. (AESL) was the consultant. AESL also provided administrative service to the research work.

This research work has provided the basis for the field data presented in this thesis. The main objective of the research program was to study alternative methods of asphaltic pavement rehabilitation, to minimize construction problems and cost, and to extend the life expectancy of the overlays (Anderson, 1983).

4.2 Test section

An experimental section was included in the research to study alternative rehabilitation methods. The experimental section was on runway 15-33 between station 3+935.00 to 4+000.00 and had a width of 20 m on each side of the centre line (Fig.4.1). This location was selected because the area was relatively flat and a uniform overlay thickness could be obtained. The test area also inhabited two major transverse cracks as well as numerous smaller sealed cracks of a blocky pattern type. Their effects on the overlay were to be studied.

Fig.4.2 is the crack map of the test section. The dotted lines indicate the length and the position of the two major cracks where treatment was required.
Fig.4.3 is a photograph taken at the south end of the test section showing a major crack where treatment prior to overlaying was required.

4.3 Existing Pavement Condition

Runway 15-33 was constructed in 1954. The pavement structure consists of 152 mm crushed gravel base course topped with 76 mm hot mix asphaltic concrete. Airfield pavement data inventory for subgrade soils indicates the soil classification to be SP, GP, SM, and SW. The textural description shows 1.0 to 1.2 m of gravel and sand over sand. Some pockets of silty sand with clay traces, and some boulders were found. The water table is about 1.2 m in the northwest of the airfield to 2.4 m in the south end of the field. The surface and subsurface drainage is fair to good. Although the airfield is not considered to be a permafrost site, indications are that silty areas are frost susceptible and severe heave has been reported on runway 09-27, with minor to moderate heave in other areas.

The freezing index for this area is about 3500 C-day and the thawing index is about 1500 C-day. The minimum air temperatures since 1954 are -49 C with an average of -45.8 C. The average maximum temperature is 23.1 C. This gives a range of about 32 C maximum in the summer to a minimum of -50 C in the winter. The annual snowfall is around 1200 mm±40 mm.

A pavement inspection reported by Transport Canada in July,1981 showed the pavement condition of Runway 15-33 as follows:

General condition:	fair to poor
Roughness:	fair
Frost heaving:	moderate in extent
	moderate in severity
Subgrade settlement:	major in extent
	moderate in severity
Asphalt surface	
Transverse cracking:	extreme in extent
	major in severity
Longitudinal cracking:	major in extent
	major in severity
Rutting:	minor in extent
	minor in severity
Ravelling:	moderate in extent
	moderate in severity

4.4 Anticipated Construction Problems

A number of problems is expected when overlaying a badly cracked pavement. Firstly, the sealant compound used to fill the cracks of the old pavement melts when it is covered by the hot overlay mix. Expansion of this sealant results in the formation of a thin layer of sealant in the vicinity of the crack at the interface of the overlay. When the overlay mix is compacted, slippage of the overlay occurs and small ridges are formed in the overlay (Fig.4.4). Secondly, the relative number of cracks which reflect to the surface of the overlay is expected to increase on a badly cracked pavement. To improve the performance of the overlay, these reflection cracks must be reduced to the minimum.

4.5 Rehabilitation Methods

Different rehabilitation methods shown in chapter 3 of this thesis had been studied in order to choose a suitable one for the test section. Three methods have been considered for the rehabilitation, they were:

1) Open graded hot mix (modified mix)
2) Geogrid and geotextiles
3) Soft grade asphalt cement overlay.

Because of the time constraints both in the preparation work and in the actual construction, two methods, namely the application of a geotextile interlayer and the use of 200-300 grade asphalt overlay, were chosen for use in the two sections of the test area.

Non-woven polyester fabric Bidim C-28, or a similar type non-woven polyester fabric, was recommended for use on the Yellowknife project. The fabric chosen, which is available locally, is known by the trade name Penroad and was labelled as a Mirafi geotextile. More information of this Penroad fabric are given in Appendix A.

The fabrics supplied were 65 m in length and 3.5, 7.0, and 9.15 m in width. Two fabric types P-50 and P-250, weighting 200 g/m^2 and 340 g/m^2 respectively, were selected for use. Two rolls of 3.5 m, one roll of 7.0 m and 9.15 m were ordered. It was later discovered that widths 7.0 m and 9.15 m were made by sewing successive 3.5 m strips together.

Fig.4.5 shows the location of the geotextile fabric in the test section. It was planned to coordinate placement of fabric with scheduled passes and width of the paver.

The 6th, 7th and last passes were planned to be paved with 200-300 pen asphaltic mix, the alternative test method of the rehabilitation. The performance of the fabric section and the soft asphalt section were then compared with the control section (pen 150-200 in the 1st and 2nd passes) to evaluate the effectiveness of the two rehabilitation methods.

Treatment to the severe cracks was done prior to the placement of the levelling course. On runway 15-33, approximately 800 m of these severe cracks was mapped as deserving treatment. The excessive sealant in cracks had to be removed before overlaying. The method developed by the contractor was to heat the surface in the vicinity of the crack with a portable propane torch and to remove the softened mixture of asphaltic concrete and sealant by hand shovelling. Fig.4.6 and Fig.4.7 show this operation.

4.6 General Description of Construction

Important aspects of the construction procedures, scheduling and problems concerning the test section are as follows:

1. The working procedures and job coordination for the test section were discussed in a site meeting on June 13th. Procedures described by Dykes(1980) served as a guideline to formulate the recommended construction procedures.
2. The application rate of tack coat, as reported in Dykes' paper, was approximately 1.13 l/m^2 of residue asphalt, and assuming a 60 percent residue for SS-1 emulsion, an application rate of 1.8 to 1.9 l/m^2 was indicated. Since the normal rate of tack coat application was 0.5 l/m^2, it was suggested that two initial passes of distributor were made at a normal rate. After the fabric was placed and rolled with a rubber tire roller, additional tack coat could be applied on the fabric surface. The amount of tack coat to be applied depended on the amount of asphalt seep through the fabric.
3. The construction schedule was so arranged that runway 15-33 could be opened for Boeing 737 to land and take off every morning.
4. The fabric was laid on June 17th, one hour after the second layer of tack coat was applied. A roll of 3.5 m width P-250 was lifted manually and was pulled and

placed manually to the specified position. Because of the lack of experience and adequate supervision, numerous wrinkles developed in this section. The second roll of 3.5 m width P-250 was also placed manually. With adequate supervision and caution, the fabric, generally speaking, was smooth and free of wrinkles.

5. Heavier 7 m and 9.15 m widths fabric were placed using a truck mounted crane to lift the roll. The free end was fixed to the pavement by hand, and the fabric was unrolled slowly by driving the crane away. The sides of the fabric were adjusted by pulling manually.
6. Plastic deformation of fabric was great. The fabric yielded when it was pulled by workmen forming winkles at the edge.
7. When the fabric was rolled by a rubber tire roller two hours after the second application of tack coat, the fabric stuck to the tires of the roller and caused wrinkles to develop (Fig.4.8). This indicated that the tack coat had not cured sufficiently and the bond between the fabric and the pavement had not developed. Rolling was stopped to allow time for the tack coat to cure. Rolling started again after the 7 m width P-250 was placed, this gives approximately 6 hours for the tack coat to cure.
8. To remove the wrinkles, standard practice required to slit them with a knife and edges overlapped with 15 cm minimum. The overlapped fabric should be tacked and

bonded together. Where 15 cm overlap width could not be achieved, another piece of fabric was patched onto it. In areas where numerous wrinkles occurred, the fabric was cut across, lifted up and relaid. It was discovered that the fabric was hard to cut and suitable tools were not available. The fabric was cut in an irregular shape and in some areas, the 15 cm overlap requirement was not satisfied.

9. Since there was not enough residual asphalt to seep through the fabric, another application of tack coat was applied on the fabric surface. This freshly sprayed tack coat made the fabric very sticky. Also the tack coat may have softened the bond between the fabric and the pavement.
10. As the truck and paver rolled over the fabric during overlaying, again the fabric was stuck to the tire and caused wrinkles and the 15 cm overlapping was lost in many areas (Fig.4.9). Unfortunately, a method to prevent fabric being picked up was only found a few days after most of the fabric had been covered.
11. Due to the necessity of paving a sufficient width of runway to allow jet aircraft operations the next morning, additional time could not be spent in correcting the wrinkles. Seams of 7 m and 9.15 m width fabric were also left untacked when the fabric was overlaid.
12. Paving was not completed until 5 am. the next morning.

As far as the working schedule was concerned, the 3rd, 4th, and 5th passes were completed in that long construction period. This was roughly equivalent to 1.5 days of normal paving plus the placement work on the fabric test section.

13. A few days later, when examining the surface of the first lift of the overlay, fine cracks were discovered in many areas over the fabric test section. A clear resilient deformation was observed when heel presure was applied in these areas. Fearing that these types of cracks would be reflected through the final surface, all these cracked areas were opened up and repaired. It was discovered that these cracks occurred at wrinkles, seams and overlapped areas of the fabric (Fig.4.10). The folds at seams were placed in such a way that there could be up to three layers of fabric. All this bulging fabric was removed and then patched up with asphaltic concrete. The remedial [illegible]rk was done before the final lift was placed.

14. The soft asphalt test section was cancelled as the result of inadequate facility. A tanker truck load of 200-300 pen grade asphalt cement arrived at site on June 18th. Since there was no heater coil in the tanker, the temperature was below the required mixing temperature. With no seperate storage facilities available, this asphalt was mixed with the 150-200 pen grade asphalt at a ratio of approximately 1 to 10. A

second tanker of 200-300 pen grade asphalt was ordered with a request for a higher delivery temperature. This load arrived at the site on June 21st, again the temperature was too low for asphalt concrete mixing. It was again combined with the 150-200 pen grade asphalt. The 6th, 7th and last passes were paved with 150-200 grade asphaltic concrete.

15. The fabric which was left in the soft asphalt section seemed to be well bonded to the pavement. However, when tack coat applied on the fabric prior to overlay, this bond was softened and the fabric again stuck to the tire of the paver. To prevent the fabric being picked up by the tire, the fabric was sanded with asphaltic concrete along the track of the paver.
16. Representative cores, 5 groups of 3, had been taken from the overlay in 5 different locations in the test area. The cores were stored at room temperature before testing.

4.7 Observation of The Construction

It is expected that the effect of the fabric will be masked by the problems arising during construction. However, the experimental project in Yellowknife still provided useful information about the construction procedures in fabric installation.

The difficulties encountered with the geotextile on the Yellowknife project was mainly caused by the inexperience in handling the fabric. Because of insufficient preparation time, many pieces of useful information could not be gathered in time. For example, the report from Sherman(1982) emphasized the importance of the type of tack coat used for sticking the fabric in place. He recommended that a penetration grade asphalt should be used instead of an emulsified asphalt.

The recommendation of using two passes of emulsified tack coat prior to the placement of the fabric seemed to be insufficient. Including the tack coat applied on the surface of the fabric, the amount of asphalt residue on P-50 and P-250 were about 0.9 l/m^2 and 1.2 l/m^2 respectively. The recommended application rate by McLaughlin (1979) was $0.8 l/m^2 \pm 0.27\ l/m^2$ and by Dykes (1980) was 1.13 l/m^2. The application rate indicated by Sherman (1982) was 1.4 l/m^2. Using Smith's (1983) method of calculation, the upper limit for the application rate for P-50 and P-250 were 1.85 l/m^2 and 2.2 l/m^2 respectively.

Curing of the tack coat was not anticipated properly before the construction. The effect of the second application of tack coat was also not foreseen. The tack coat that was applied on the fabric surface apparently broke the bond between the fabric and the pavement. As a result, the fabric stuck to the tires of the roller, trucks and paver and developed numerous wrinkles.

Difficulties in placing the fabric were not foreseen in the job planning stage. The installation was further complicated by the unexpected fabric seams found in the 7 m and 9 m width fabric. Time constraint for overlay construction was another problem. The requirement of having the runway open for operation the next day prevented proper construction procedures and corrective measures being taken when problems arose. Wrinkles were left unrepaired, seams were left unfixed and untacked and triple layers resulted. These caused sponginess and cracks to appear in the overlay surface. As an afterthought, the location should have been out of the main runway so that when problems arose, more time could have been spent in the remedial work, especially for the case when construction problems were not anticipated due to insufficient knowledge of the test material and inadequate experience with the construction procedures.

The fabric used was not a suitable material. High inelastic elongation of the fabric at low tensile stress made it difficult to install. Wrinkles developed at the edges when the fabric was pulled by workers in placing the fabric. This elongation also caused wrinkles to develop when the fabric was pulled up by the tires of the trucks and paver. Besides, fabric with sewn seams was difficult to handle and therefore should be avoided in future projects.

The removal of excessive sealant by using the propane torch was effective. No ridges were found in the overlay directly above the treated cracks. However, it is hard to determine which crack should be treated since ridges are not only found in large cracks but in smaller cracks as well.

Fig.4.4 shows a location where ridges were formed above relatively small cracks. Fabric was effective in preventing the sealant from coming up through the asphaltic-concrete overlay, and no ridges were found in the fabric test section. Fig.4.11 shows the sealant being absorbed and attached to the fabric.

4.8 Performance of The Test Section

A crack survey of the test section done on 4th June, 1984 is shown in Fig.4.12. About 30 percent of the cracks reflected to the surface of the overlay after the first winter. There appears to be no significant difference between the test section and the control section, nor between cracks that have been pre-treated and those that have not. The report by Anderson et al.(1984b) stated that

> Discontinuous transverse reflection cracks spaced roughly 5 m. apart have occurred in the test section. Similar longitudinal cracks are present on a somewhat closer spacing. The resultant pattern is not obviously different from adjacent pavement areas containing no fabric.
>
> Some of the longitudinal cracks had developed a braided pattern which resulted in occasional break-outs. There was a lower incidence of pavement break-outs along cracks in the test section, but this phenomenon was not sufficiently common over the entire runway area to permit a confident comparison.

4.9 Summary

A summary of observations drawn from the experimental project are:

1. The problems arising during construction will affect the results of the experimental section. Experience

gained from this project may be useful in improving fabric placement procedures.

2. The application of tack coat is a important construction procedure. The amount of tack coat applied, the type of tack coat used, the amount of curing time allowed have to be determined for proper installation of the fabric.
3. Fabric with high plastic deformation at low tensile strength may create construction difficulties.
4. Fabric seams are difficult to tack and they create spongy effect which cause the overlay to crack. Fabric which formed by sewing pieces together should be avoided.
5. The fabric is successful in preventing the crack sealant from coming up through the asphaltic concrete overlay and preventing ridges forming in the overlay.
6. Selective crack repair using a propane torch and techniques developed on this project is effective in eliminating excessive crack sealant. This method does not appear effective in reducing the cracking of the overlay.
7. Under the particular conditions being described, the fabric seems to have little effect on the prevention of cracking. About 30 percent of the cracks reflected to the surface both in the fabric section and in the control section after the first winter of service.

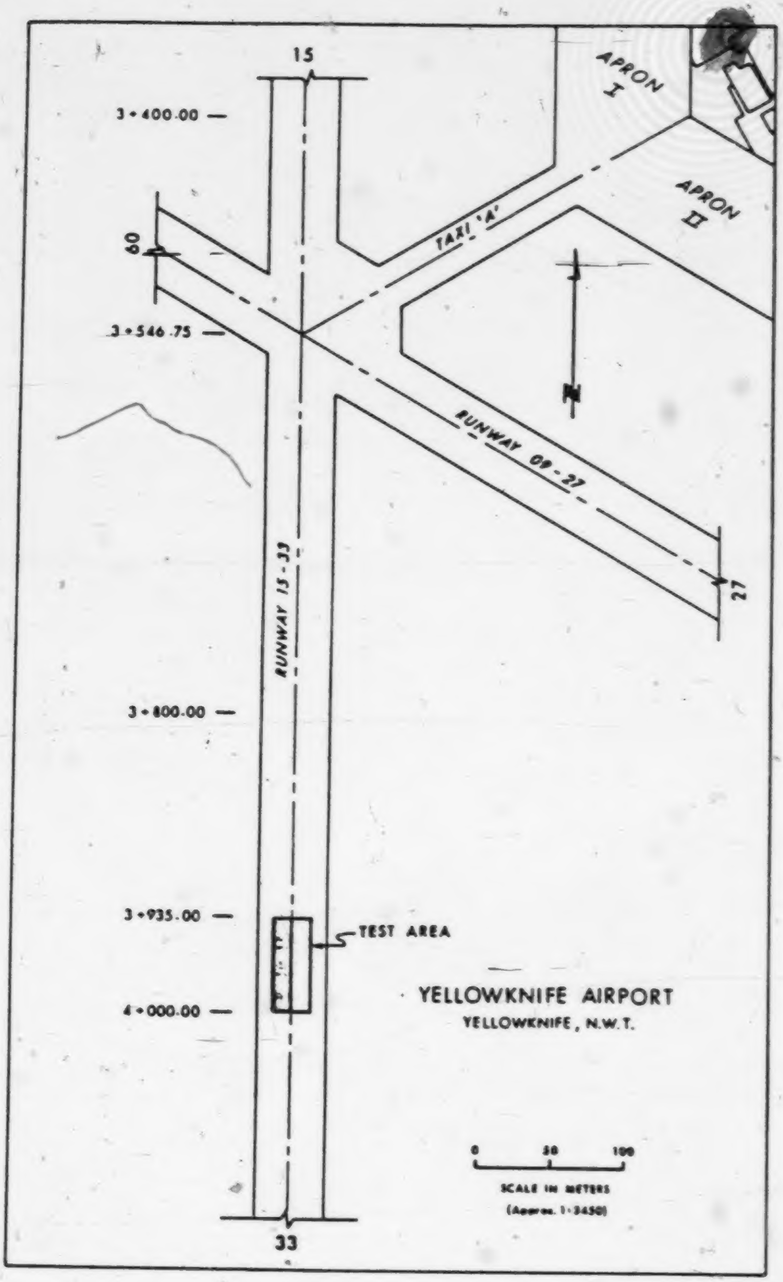


FIG. 4.1 LOCATION OF TEST AREA ON RUNWAY 15-33

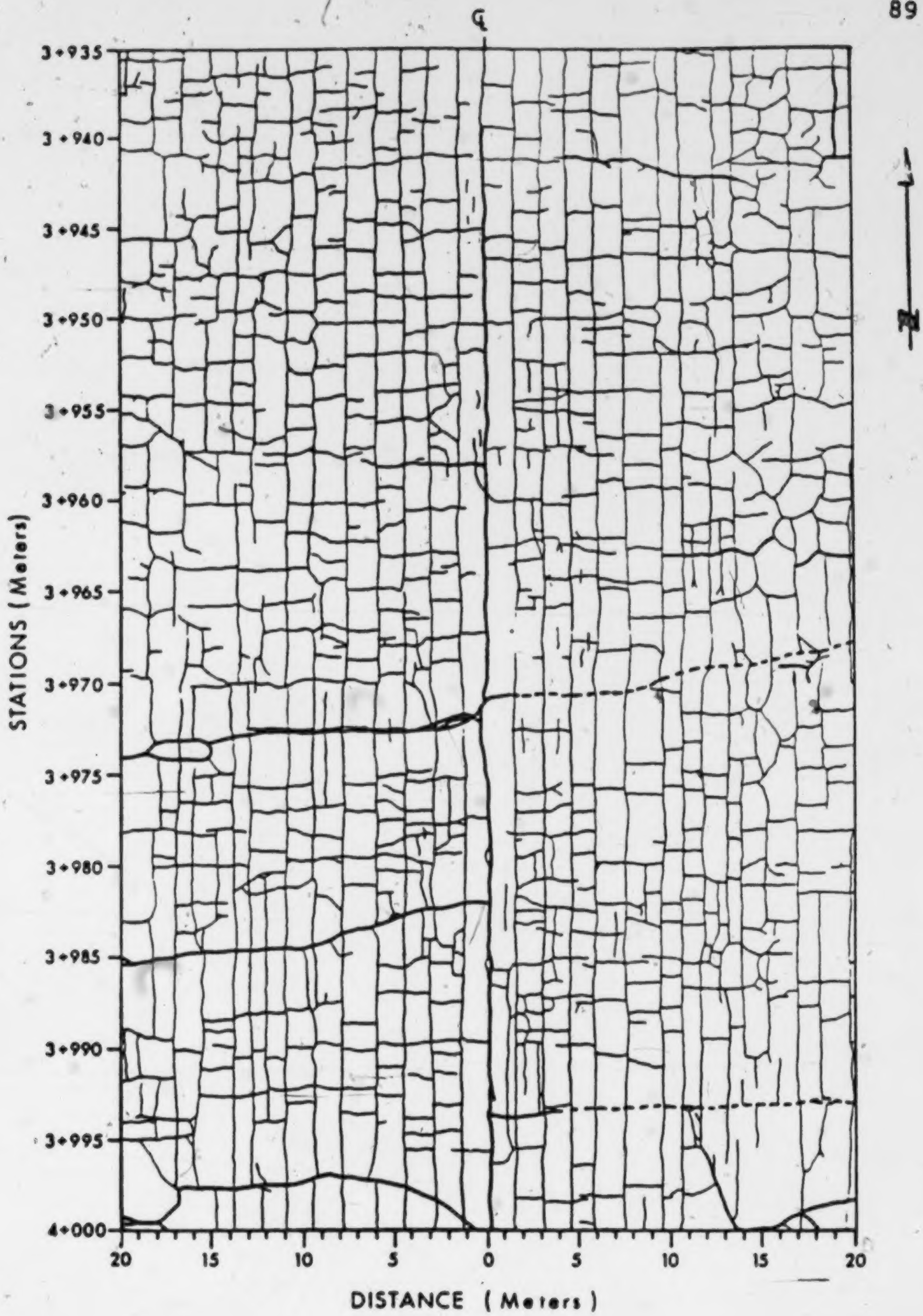


FIG. 4.2 CRACK MAP OF TEST AREA BEFORE OVERLAYING

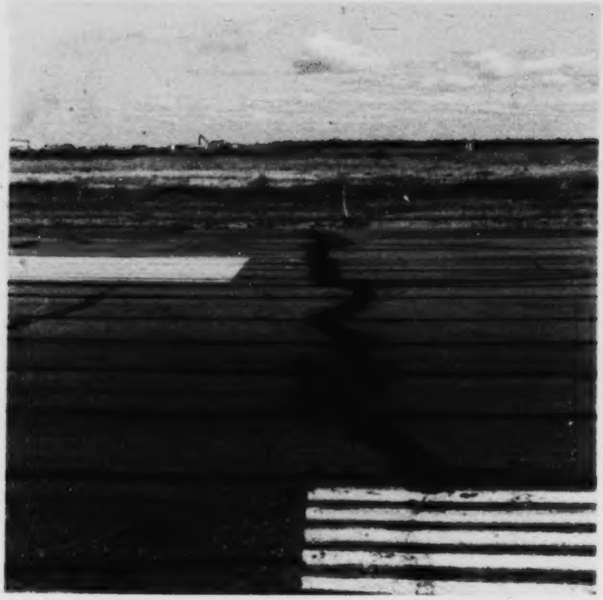


FIG. 4.3 MAJOR CRACK AT SOUTH END OF THE TEST AREA

FIG. 4.4 SMALL RIDGES FORMED IN THE OVERLAY

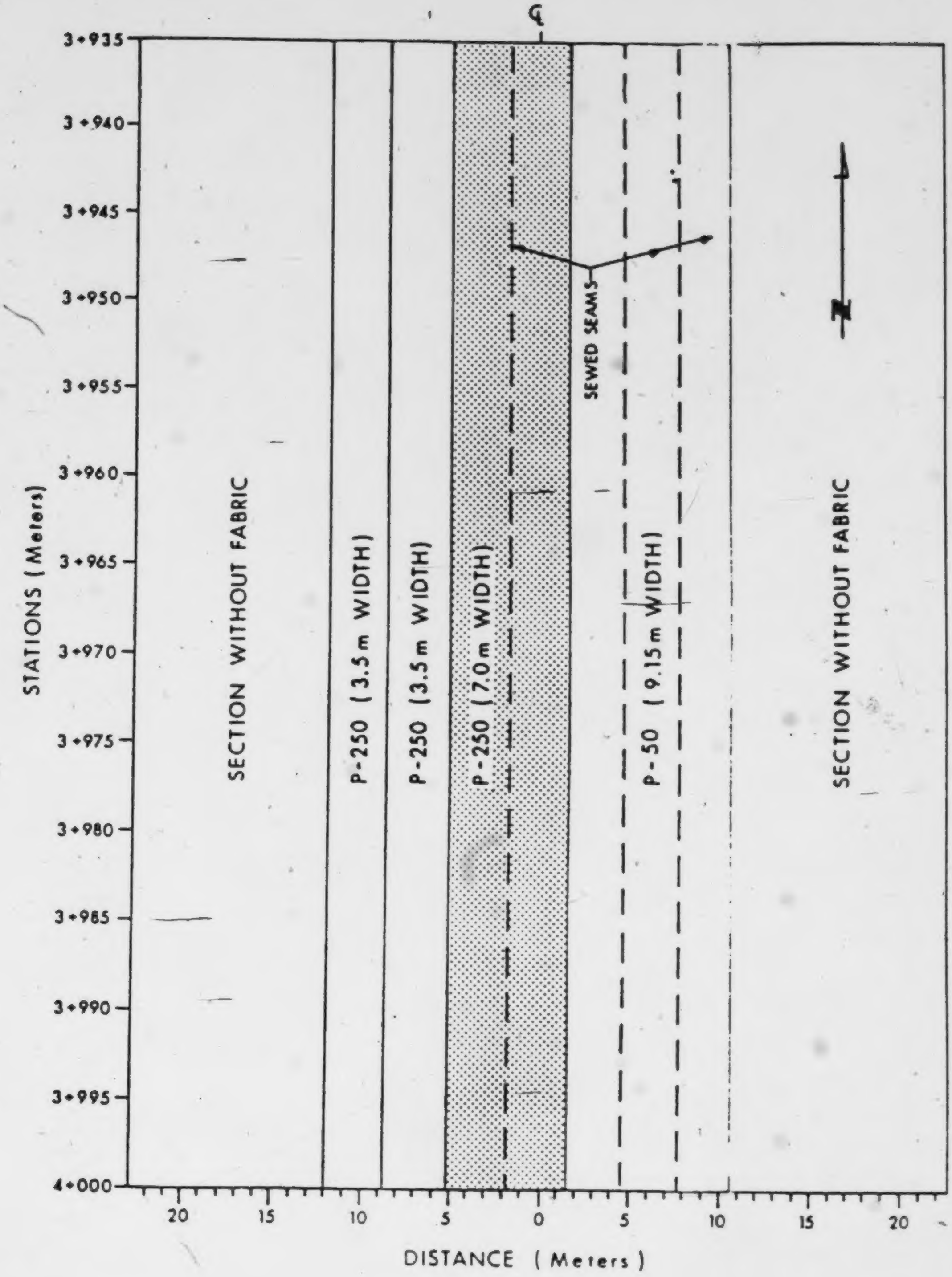


FIG. 4.5 LOCATION OF THE FABRICS IN THE TEST AREA

FIG. 4.6 PORTABLE PROPANE TORCH HEATING THE CRACK SURFACE

FIG. 4.7 REMOVING SOFTENED MIXTURE BY HAND SHOVELLING

FIG. 4.8 FABRIC SURFACE AFTER ROLLING

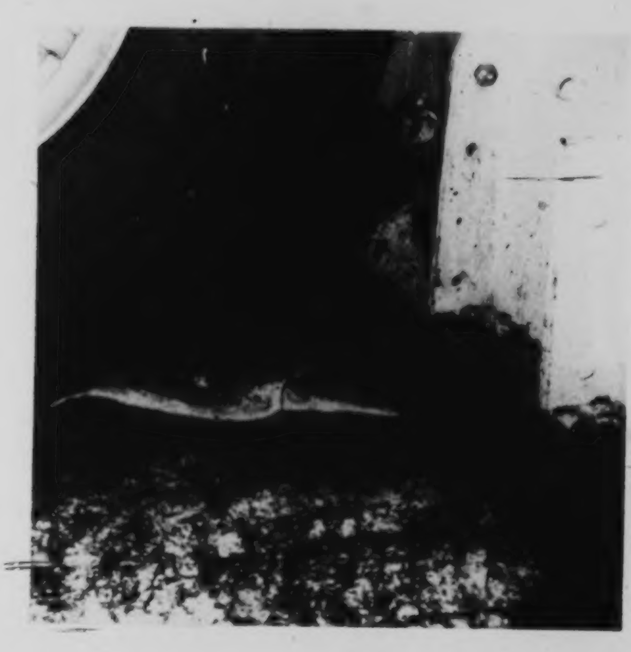


FIG. 4.9 TIRE PICKING UP THE FABRIC

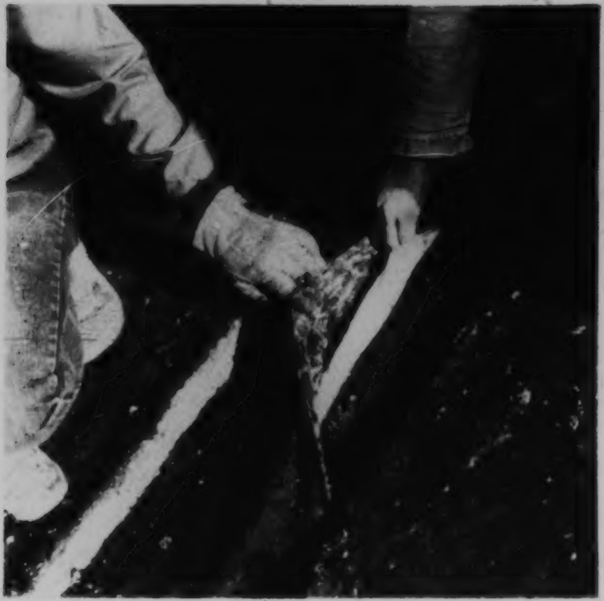


FIG. 4.10 THE UNTACKED SEAM

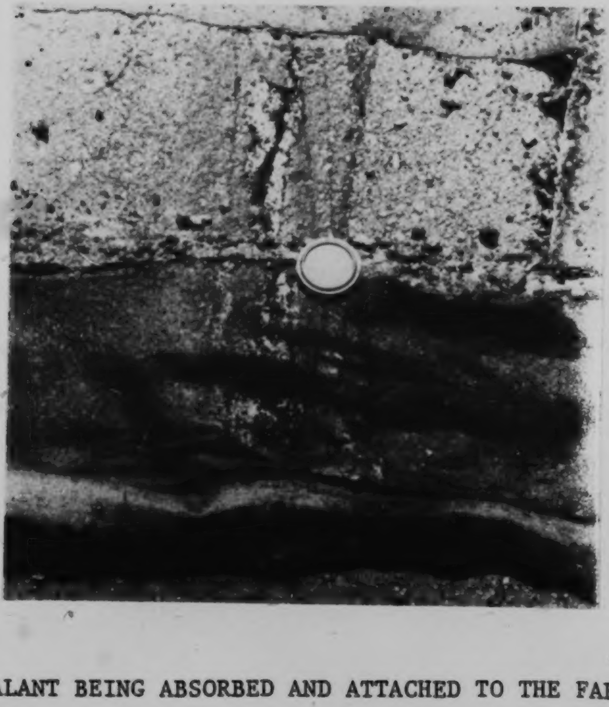


FIG. 4.11 SEALANT BEING ABSORBED AND ATTACHED TO THE FABRIC

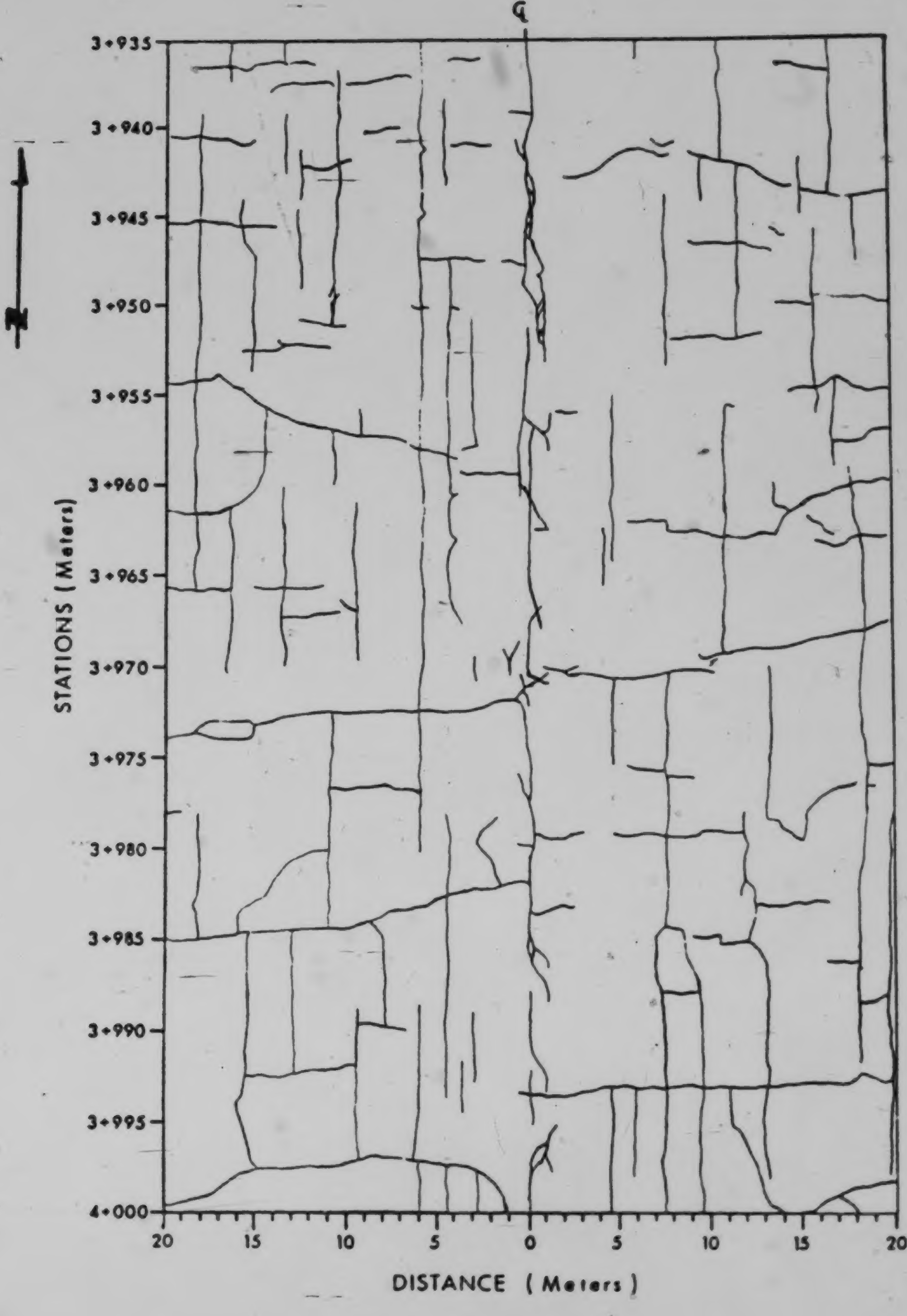


FIG. 4.12 CRACK MAP OF TEST AREA AFTER ONE WINTER

CHAPTER 5

TESTS AND ANALYSIS

5.1 Objectives of the Test Program

The first objective of the test program was to determine the material characteristics of the asphaltic concrete used in the Yellowknife overlay. The low temperature properties of the asphaltic concrete, i.e. tensile strength, failure strain and failure stiffness, were determined. The properties of the recovered asphalt, such as penetration, viscosity and temperature susceptibility, were also found. The second objective of the test program was to determine the cracking potential of the overlay in the winter of 1983-1984 based on the properties of the cores tested.

5.2 The Laboratory Test Program

Fifteen cores, 100 mm diameter asphaltic concrete cylinders, were obtained from the overlay at the time of construction and were transported to the laboratory and stored at room temperature. Cores with an irregular surface were sawed to obtain a constant thickness and a flat surface for mounting gauge points. Out of the five groups of three samples each, one group of the samples were too thin for testing. The samples were broken during storage and these samples were discarded.

The tensile splitting test method (Christianson, 1970) was used to characterize the tensile properties of the asphaltic concrete. With regard to the ambient temperatures that the overlay encountered during the winter of 1983-1984, testing temperatures were selected to be 20 F, 0 F, -10 F and -20 F (-6.7 C, -17.8 C, -23.3 C and -28.9 C respectively). One group of three cores was used for each testing temperature. After the test, the asphalt was extracted from the broken cores by a commercial laboratory. The properties of the recovered asphalt were then tested and analyzed. The laboratory tests besides the tensile splitting test included:

a) Test on undisturbed core samples

-----Bulk specific gravity (ASTM D2726)

b) Tests on broken cores after splitting

-----Asphalt extraction (ASTM D2172 Method A)

-----Asphalt recovery from solution by Abson Method (ASTM D1856)

c) Tests on recovered asphalt

-----Penetration at 4 C and 25 C (ASTM D5)

-----Absolute Viscosity at 60 C (ASTM D2171)

-----Kinematic Viscosity at 135 C (ASTM D2170)

-----Ring and Ball softening point (ASTM D36)

5.3 Tensile Splitting Test

The theory involved in the tensile splitting test and the proper procedures in conducting this test are described in theses of Gillespie (1966), Christison (1966), Hahn (1967) and Christianson (1970) and therefore are not repeated here. There is some modification in the testing apparatus used in this test program. A detailed description of the test apparatus is found in Appendix B.

Briefly, the tensile splitting test consists of loading a 100 mm diameter asphaltic concrete cylinder via a pair of steel loading strips across the diameter in a compression testing frame in a temperature controlled chamber. The temperature of the chamber is kept at a constant temperature which is specified in the test program. The nominal loading rate was chosen to be 0.06 in/min (0.0254 mm/sec). A change in the loading rate as the result of the slackness in the component parts of the loading machine was reported by Christison (1966). A Linear Variable Differential Transducer (LVDT) was introduced to monitor this loading rate.

As the sample is compressed, a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametrical plane is formed which ultimately causes the sample to fail by splitting along the vertical diameter. After splitting, the sample was examined to observe any unusual occurrences which may be used to explain the results obtained.

The applied load and the horizontal deformation are monitored through a load cell and two alternative current type LVDTs mounted horizontally on each faces of the sample in the centre at 25.4 mm apart. The loading rate is monitored by a direct current type LVDT attached to the lower loading plate of the loading frame. Output signals from these gauges are transmitted to another room and recorded on a six channels X-Y plotter. By calculating from these records, the tensile strength, the strain and the stiffness modulus of the sample at the specified loading rate and test temperature can be found.

The tensile strength across the sample at any time, by Frocht's (1948) formula, is equal to:

$$\sigma_x = \frac{2P}{\pi dt} \tag{5.1}$$

where σ_x is the tensile strength in psi.

P is the applied load in pounds

t is the thickness of the sample in inches

d is the diameter of the sample in inches

The number of inches measured in the horizontal LVDTs is the total strain at the centre since the LVDT is one inch (25.4 mm) apart. The part of strain due to the tensile stress is calculated to be equal to one half of the total strain assuming the Poisson's ratio of the asphaltic concrete is 0.33. The stiffness of the mix is then equal to the ratio of tensile stress to the tensile strain computed.

5.4 The Computer Analysis

As mentioned in Section 2.3 of Chapter 2, cracking of an asphaltic concrete overlay occurs when its stresses exceed the tensile strength of the material. Using this theory, Christison (1972) developed a computer based prediction model for low temperature cracking. The program was later modified under the National Cooperative Highway Research Program Project 1-10B by Woodward-Clyde Consultants, San Francisco (Finn et al.,1976b) to arrive at the damage prediction model ---- the COLD (Computation of Low-Temperature Damage) program. This program had been used to predict the cracking potential of the pavement in test projects in Alberta and Manitoba, and in western Texas (Anderson and Epps, 1983). The prediction has been compared to the observed cracking at these sites and the correlation is sufficiently good to indicate the acceptability of the COLD program. The underlying theory of this program can be found in the thesis of Christison and is not going to be discussed in detail in this thesis.

The COLD Program can be divided into two main components. The first part of the program calculates pavement temperatures from the air temperature and the solar radiation data, and the second part of the program calculates the induced thermal stresses in the pavement in relation to the pavement temperatures calculated in the first part of the program. The thermal stresses in the pavement can be computed using either the pseudo-elastic beam analysis or the pseudo-elastic slab analysis depending on the boundary constraints on an element of the pavement structure under analysis. The beam analysis implies the conditions

$$\epsilon_x = 0 \ , \ \sigma_y = \sigma_z = 0 \tag{5.2}$$

$$\sigma_x = - \int_{t_0}^{t} S(\Delta t,T) \ \alpha(T) \ d(T)_t \tag{5.3}$$

and the slab analysis implies the conditions

$$\epsilon_x = \epsilon_y = 0 \ , \ \sigma_z = 0 \tag{5.4}$$

$$\sigma_x = - \int_{t_0}^{t} \frac{S(\Delta t,T) \ \alpha(T)}{1 - \nu(T,t)} \ d(T)_t \tag{5.5}$$

The beam assumptions approximate conditions at the edge of a pavement while the slab assumptions are approximate to the conditions in the centre of a pavement. For simplicity, the overlay is analyzed in the slab condition only.

It was found in Christison's study that the maximum tensile stress occurs at or near the upper surface. The stress at the 12.5 mm depth was adopted as a critical stress condition.

5.5 Input Data of the COLD Program

A description of the program COLD is presented in a unpublished report (Finn et al.,1976c). The appendix of the report includes a user's manual detailing input requirements of the COLD program. The pavement temperatures, from which the induced stress is calculated, are estimated from the air temperatures and the solar radiation values. These data were obtained from the Atmospheric Environment Service of the Government of Canada. Air temperature is recorded every hour in the weather station at Yellowknife airport. Since radiation values are not available for the Yellowknife station, radiation records from the nearby station at Fort Smith, N.W.T. were used instead.

Temperature records of Yellowknife for the winter of 1983-1984 were studied. From these records, a period of 20 days including the day of the lowest recorded temperature during that winter was selected for the analysis. Data of the temperature and the radiation for that analysis period are in Appendix C.

Stiffness modulus of the mix is estimated from the recovered asphalt properties using indirect nomograhic procedures described in Chapter 2. A loading time of 7200 seconds was adopted in the estimation as this was found to be appropriate for conditions in Manitoba.

Tensile strength values of the asphaltic concrete at different temperatures are required in the program for comparison to the thermal stress, thereby enabling a prediction of cracking.

The properties of the pavement component layers are estimated for input to the program. These data include:

1) The temperatures at which freezing commence in the base and subgrade,
2) The temperatures at which a percentage of water in the

base and subgrade are frozen,

3) The thermal properties of the surface mix: the absorptivity, the emissivity and the convection coefficient,

4) The thermal properties of each layer: the unfrozen and frozen thermal conductivity, the unfrozen and frozen heat capacity of each layer,

5) The dry density and the moisture content of each layer,

5) The percentage of water frozen in each layer,

6) The coefficient of expansion of asphaltic concrete,

7) The Poisson's ratio of the asphaltic concrete.

Table 5.1 list the input values for the above mentioned properties of the pavement layers.

TABLE 5.1 INPUT VARIABLES FOR THE COLD PROGRAM

Meteorological		Hourly Air Temperature (F) Hourly Solar Radiation (Langleys/hr)		
Structural		Number of Layers = 3		
		Asphaltic Concrete	Base	Subgrade
	Thickness (in)	6.0	6.0	60.0
Physical	Dry Density (pcf)	148.0	138.0	112.0
	Moisture Content (% by dry wt.)	1.0	5.0	15.0
Thermal	Conductivity (Btu/hr/ft/F)			
	Unfrozen	0.84	0.51	1.34
	Frozen	0.84	1.41	1.91
	Heat Capacity (Btu/lb/F)			
	Unfrozen	0.22	0.21	0.24
	Frozen	0.22	0.18	0.19
	Freezing Zone		32F-30F	32F-30F
	% Moisture Frozen		99.0	99.0

CHAPTER 6

PRESENTATION AND DISCUSSION OF RESULTS

6.1 Tensile Splitting Test

The results obtained from the tensile splitting tests are summarized in Table 6.1. Although the nominal loading rate was at 0.06 in/min (0.0254 mm/sec), the actual loading rate ranged from 0.062 in/min to 0.071 in/min (0.026 mm/sec to 0.030 mm/sec). The table also shows the failure time, the tensile stress at failure, the total horizontal strain for the centre 25.4 mm at failure, and the tensile stiffness modulus for each individual sample.

Fig.6.1 to 6.4 show the stress versus strain plots for the samples at different test temperatures. The scales are changed for each plot to accomodate the data. As seen in these figures, some samples show abnormally high initial strain. This discrepancy may be due to the tapering of the cylindrical samples. As the samples were cored from pavement, the cores may not be a perfect cylindrical shape. When these cores were loaded under the loading strips, one side of the sample might be loaded while the other side was still not loaded. This uneven loading might cause the high strain in the initial stage. The figures also show that unexpectedly there are cases where samples are very brittle, such as samples 2-1 and 3-1, and cases where samples have the tensile strength very different from that of the group, such as samples 1-3, 3-1 and 4-2. All these show the difficulty of the test to measure low temperature properties. It is unknown whether these are due to the variability of properties of asphaltic concrete or of the test itself. Normally a minimum of five samples are used for each loading condition.

Fig.6.5 shows the failure stress of the samples related to the testing temperature. The average failure stress was also plotted. The tensile strength (failure stress) of the asphaltic concrete increases with the decrease in temperature. It reaches ~~the~~ highest stress at about -14 F (-25 C) with a magnitude about 360 psi (2490 kPa), and then the strength decreases with further decrease in temperature.

Fig.6.6 shows the failure strain of the samples related to the testing temperature. The average failure strain of the samples was also plotted. The figure shows that the asphaltic concrete can withstand more strain at higher temperatures. The asphaltic concrete becomes more brittle and breaks at less strain at lower temperatures. The decrease in average failure strain reaches the lowest level at about -12 F (-24 C). Further decrease in temperature has little effect on the failure strain of the asphaltic concrete.

Fig.6.7 shows the failure tensile stiffness modulus of the samples related to the testing temperature. The failure tensile stiffness modulus is computed by considering the ratio of the tensile stress at failure to the tensile strain. The tensile stress is the average stress at the centre 25.4 mm between the two gauge points. The tensile strain is the part of the horizontal strain resulted from this tensile stress assuming a Poisson's ratio of 0.33. The average stiffness modulus of the samples was also plotted. This figure reflects the changes that occur jointly in the failure stress and failure strain.

6.2 Recovered Asphalt Cement Tests

Table 6.2 shows the test results of the properties of the asphalt recovered from cores of group two, three and four. Individual results, as well as the average, are presented in view of the dispersion in some of the results.

Despite the original asphalt cement grade of 150-200 being used in the overlay construction, tests on the recovered asphalt show it to be a hard asphalt. Penetration at 25 C ranges from a low of 46 dm to a high of 60 dm. A retained penetration of 23 to 40 percent indicates that excessive hardening seems to have occurred during the construction, although test results on the original asphalt cement are not available.

Viscosities measured at 60 C and 135 C also reveal the hardness of the asphalt. The viscosity at 60 C ranges from 3976 to 5760 poises and the viscosity at 135 C ranges from 495 to 619 centistokes.

The PI of the asphalt was estimated using the Shell method and the Bitumen Test Data Chart (Kopvillem and Heukelom,1969). The average values of penetration and viscosity were used in the estimation. Fig. 6.8 shows the plotted values and indicates that the asphalt is a straight run asphalt. The PI is equal to 0.0 .

The average density of the samples is about 2224 kg/m^3. The average asphalt content of the samples is about 5.0 percent. The average air voids of the samples is about 7.0 percent (Table 6.3). For the upper course of the overlay, the density and compaction records in the engineer's report show that the density is about 2280 kg/m^3. The asphalt content is about 4.9 percent. The air voids of the upper course was then estimated to be 5.8 percent. Sample calculations are shown in Appendix D.

As mentioned in Section 5.4, stress at the 12.5 mm depth is critical in predicting the cracking, therefore the stiffness modulus of the asphaltic concrete of the upper course is required. The stiffness modulus of the upper course was estimated using the method of Bonnaure et al.(1977) with the air voids of the mix assumed to be 5.8 percent. The stiffness moduli of mix at different temperatures are shown in Table 6.3. These values were input into the COLD program for estimating the cracking potential of the overlay.

6.3 Cracking Potential Analysis

Fig.6.9 and Fig.6.10 show the results of the COLD program analysis. The figures plot the input air temperatures, the calculated pavement temperatures, the input tensile strength of the mix and the calculated induced thermal stress in the overlay. The study covered the coldest period of the 1983-1984 winter from January 16 to February 5. Examination of the temperature input data in Appendix C shows that the maximum rate of temperature change is 5.4 F/hr (3 C/hr).

On Jan 25, the highest air temperature was -31 F (-35 C) and the lowest air temperature was -42 F (-41 C). The calculated thermal induced stress exceeded the tensile strength of the mix when the air temperature was below -36 F (-38 C). On Jan 26, the air temperature dropped to the lowest at -43.6 F (-42 C). The thermal stress at this temperature was 494.6 psi (3400 kPa), exceeding the tensile strength of the mix by 194.4 psi (1340 kPa). On Feb 4, the thermal stress again exceeded the tensile strength of the mix when the air temperature fell to -36 F (-38 C).

The pavement is expected to crack if the induced thermal stress exceeds the tensile strength of the asphaltic concrete. This analysis shows that the overlay will crack at ambient temperatures below -38 C. The temperature records from the Atmospheric Environment of the Governemnt of Canada show that there were twelve days in which the daily minimum temperature fell below the above mentioned critical temperature. Therefore it is not surprising to find that the overlay was cracked after only one winter.

Using the criteria of limiting stiffness of asphalt, Gaw (1978) suggested that the temperature at which a mix reaches the limiting stiffness of 1.45 x 10^5 psi. (1 x 10^9 Pa.) at 1800 seconds loading time is the expected cracking temperature. With this method, the estimated cracking temperature of the overlay comes to about -52 C.

Since both the COLD program analysis and the asphalt limiting stiffness method predict the cracking based on thermal effects only, using these methods to predict the cracking potential of the overlay may be overly simplified. Both methods have not accounted for possible effects of stress concentration, the base and the subgrade shrinkage, the traffic loading and the freeze and thaw action. As a result, the actual cracking temperature may well have been higher than the predicted cracking temperature.

TABLE 6.1 SUMMARY OF TEST RESULTS OF THE TENSILE SPLITTING TEST

TESTING TEMPERATURE	20 F			0 F			-10 F			-20 F		
GROUP	ONE			TWO			THREE			FOUR		
SAMPLE	1-1	1-2	1-3	2-1	2-2	2-3	3-1	3-2	3-3	4-1	4-2	4-3
THICKNESS OF CORE (mm)	33.3	28.6	27.0	36.5	31.0*	27.0*	34.9	31.8	34.9	45.2	48.4	44.5
DENSITY OF CORE (kg/m^3)	2,196	2,220	2,173	2,260	2,231	2,224	2,236	2,251	2,258	2,251	2,258	2,236
TIME TO FAILURE (sec)	151	140	135	105	96	109	90	77	109	89	93	104
LOADING RATE (in/min)	0.068	0.068	0.063	0.068	0.071	0.062	0.065	0.063	0.067	0.068	0.064	0.065
TENSILE STRESS AT FAILURE $(kPa)\times10^3$	1.135	1.168	0.715	2.219	2.327	1.958	2.039	2.763	2.669	2.603	1.923	2.823
TOTAL STRAIN AT FAILURE $(in/in)\times10^{-3}$	6.13	5.21	11.50	0.10	2.73	2.06	0.53	1.08	2.95	1.98	1.03	2.28
STIFFNESS MODULUS $(kPa)\times10^5$	3.38	4.10	1.13	124.60	15.55	17.34	69.25	46.66	16.50	23.98	34.06	22.64

* Indicates the thickness of the sample after sawed

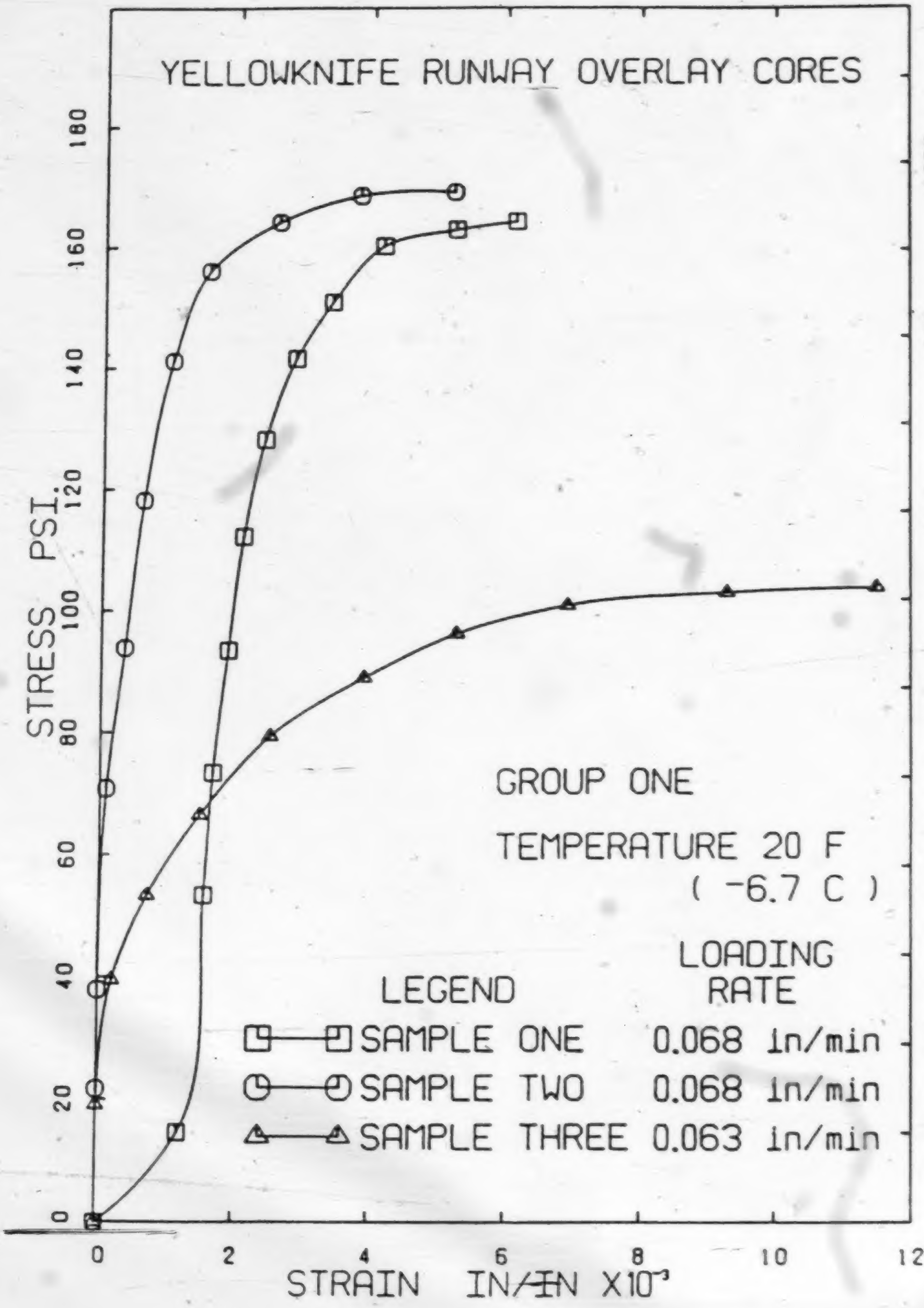


FIG. 6.1 STRESS-STRAIN RELATIONSHIP, GROUP ONE

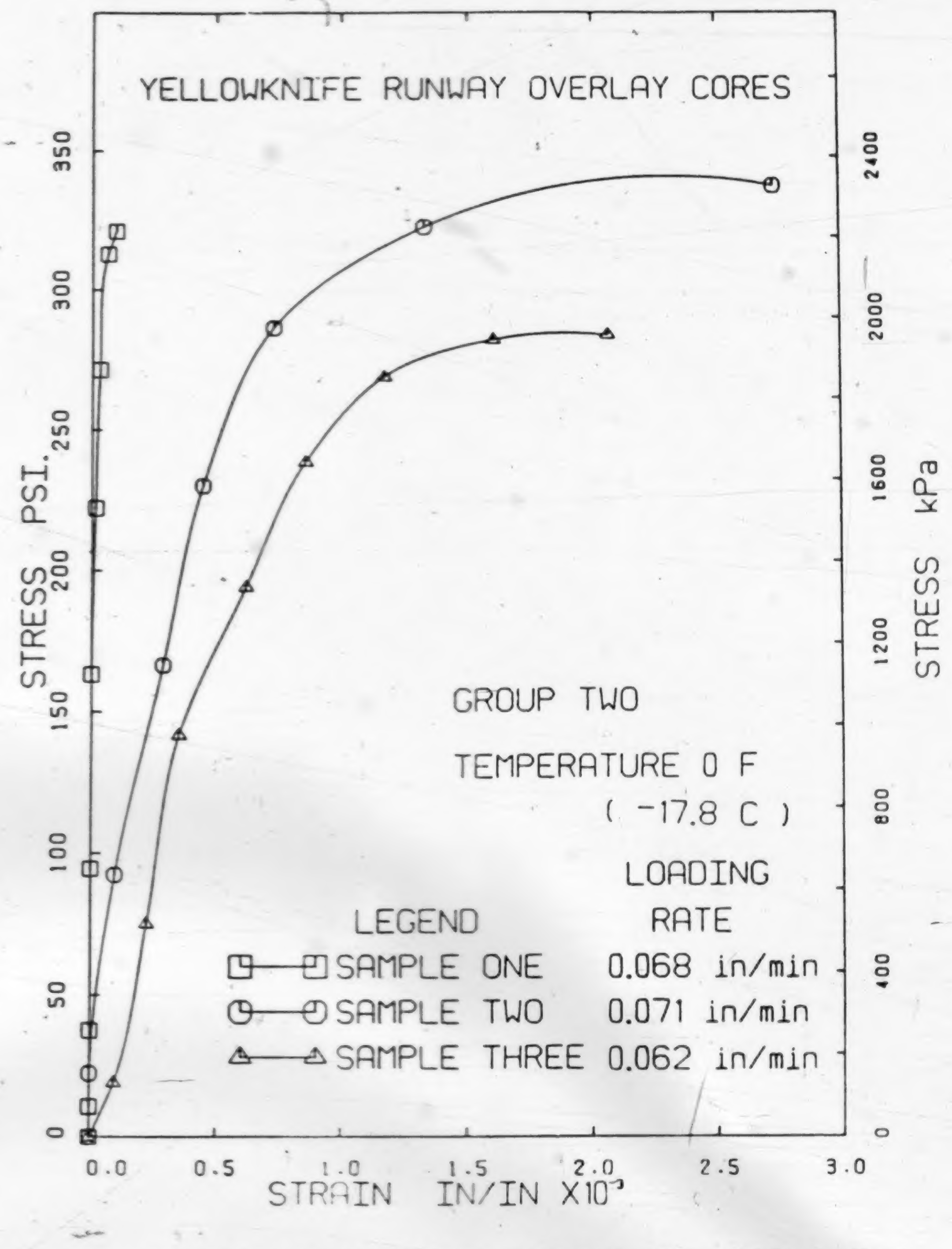


FIG. 6.2 STRESS-STRAIN RELATIONSHIP, GROUP TWO

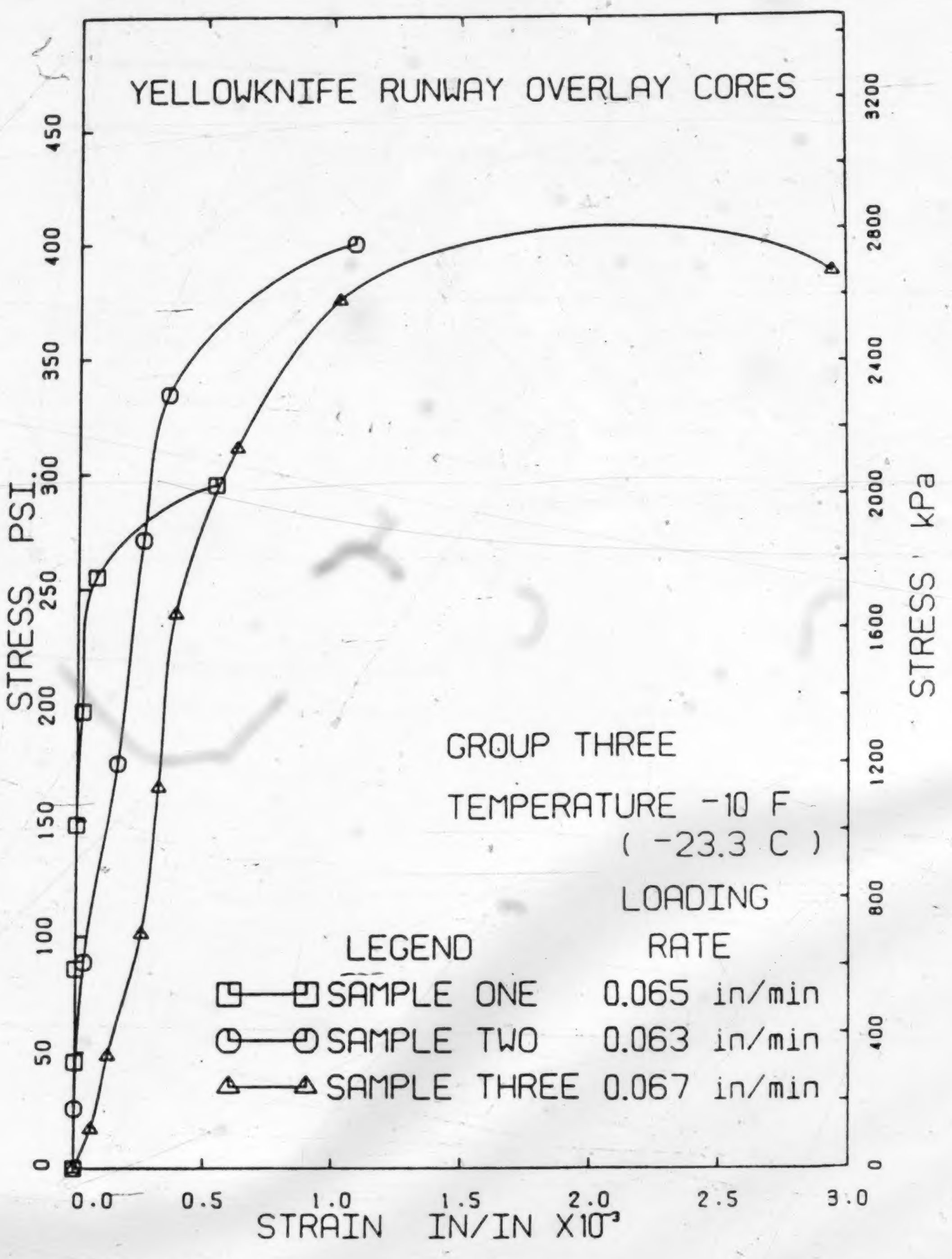


FIG. 6.3 STRESS-STRAIN RELATIONSHIP, GROUP THREE

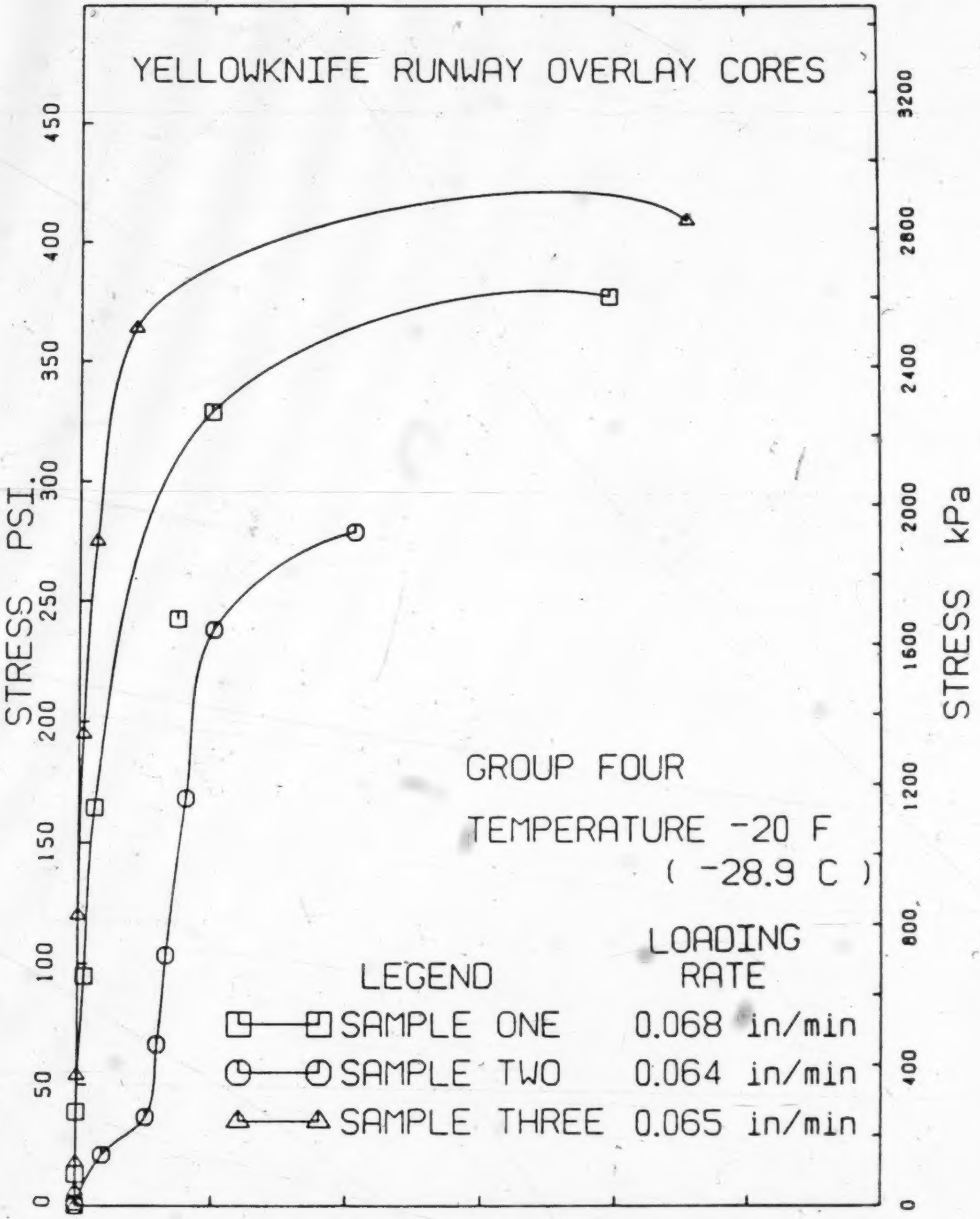


FIG. 6.4 STRESS-STRAIN RELATIONSHIP, GROUP FOUR

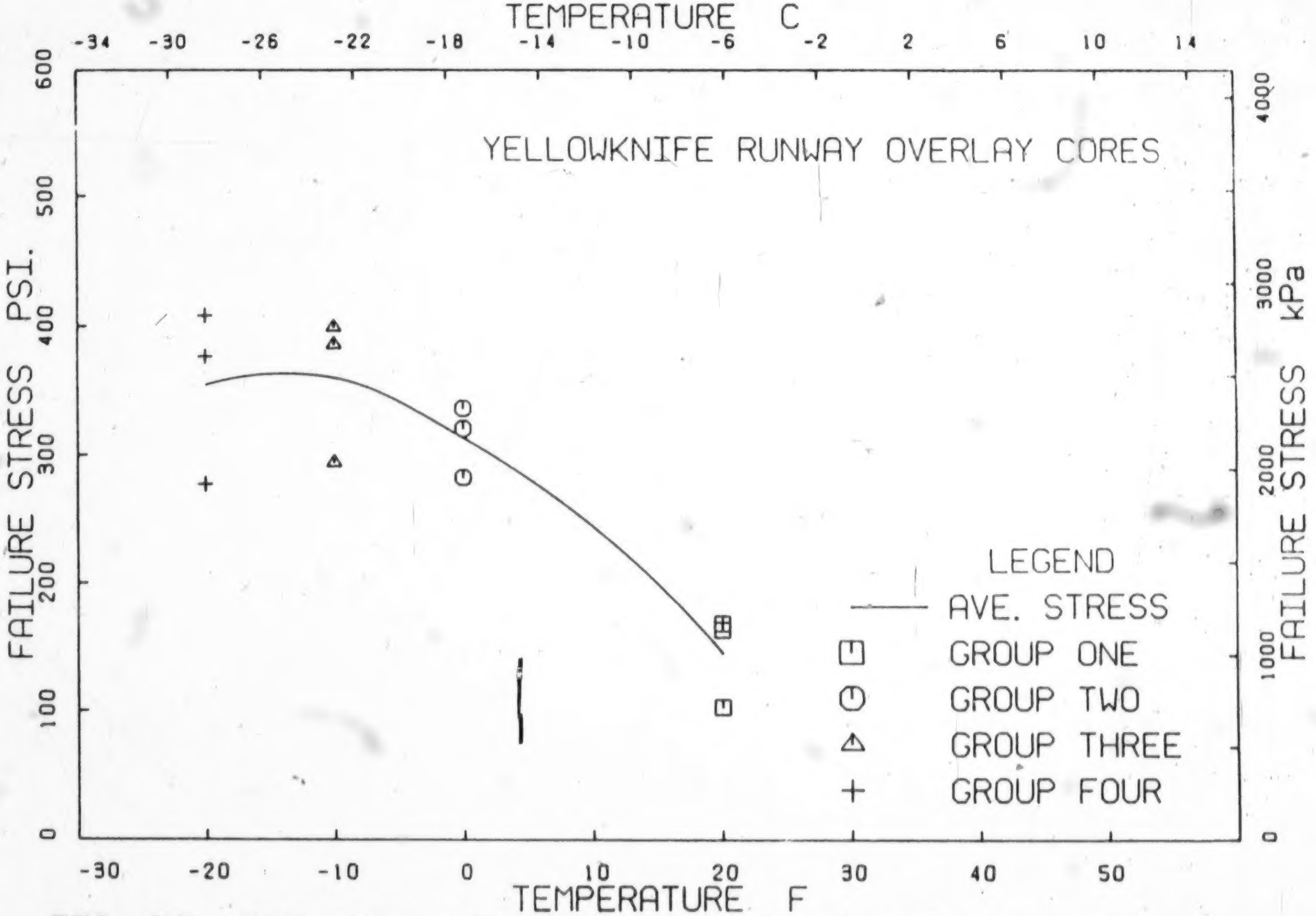


FIG. 6.5 FAILURE TENSILE STRESS-TEMPERATURE RELATIONSHIP

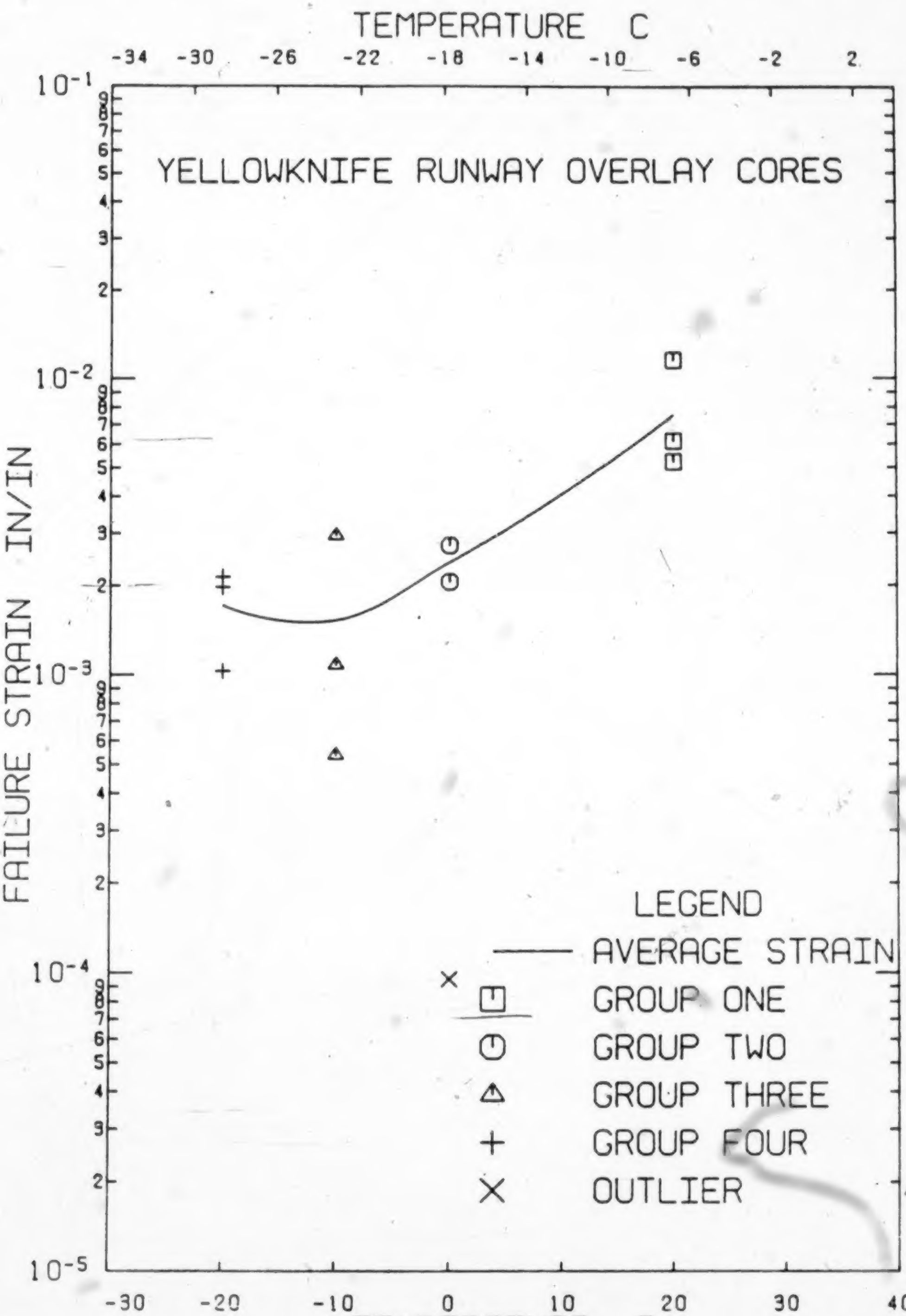


FIG. 6.6 FAILURE STRAIN-TEMPERATURE RELATIONSHIP

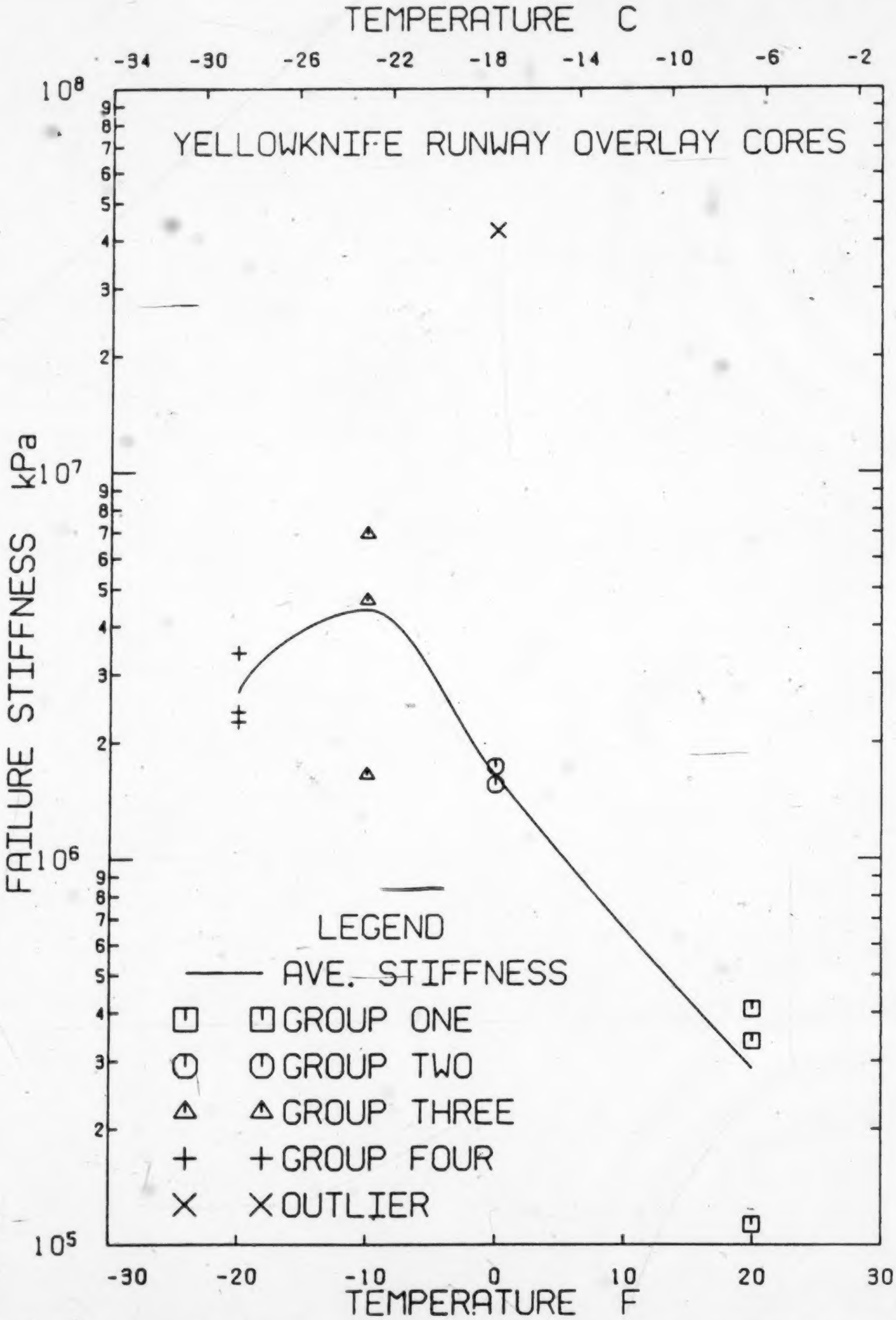


FIG. 6.7 FAILURE TENSILE STIFFNESS-TEMPERATURE RELATIONSHIP

TABLE 6.2 SUMMARY OF RECOVERED ASPHALT CEMENT PROPERTIES

	Sample Groups			
Characteristics	Group 2	Group 3	Group 4	Average
Penetration, dm				
@ 4 C 200 g, 60 s	29	26	23	26
100 g, 5 s	9	8	6	8
@ 25 C 100 g, 5 s	60	53	46	53
Viscosity,				
@ 60 C, poise	5760	5143	3976	4960
@ 135 C, cSt.	588	619	495	567
Softening Point, R&B in degree C	52.0	53.0	51.5	52.0
Penetration Index				0.0
Asphalt Content % by wt. of mix	5.3	5.0	4.9	5.0

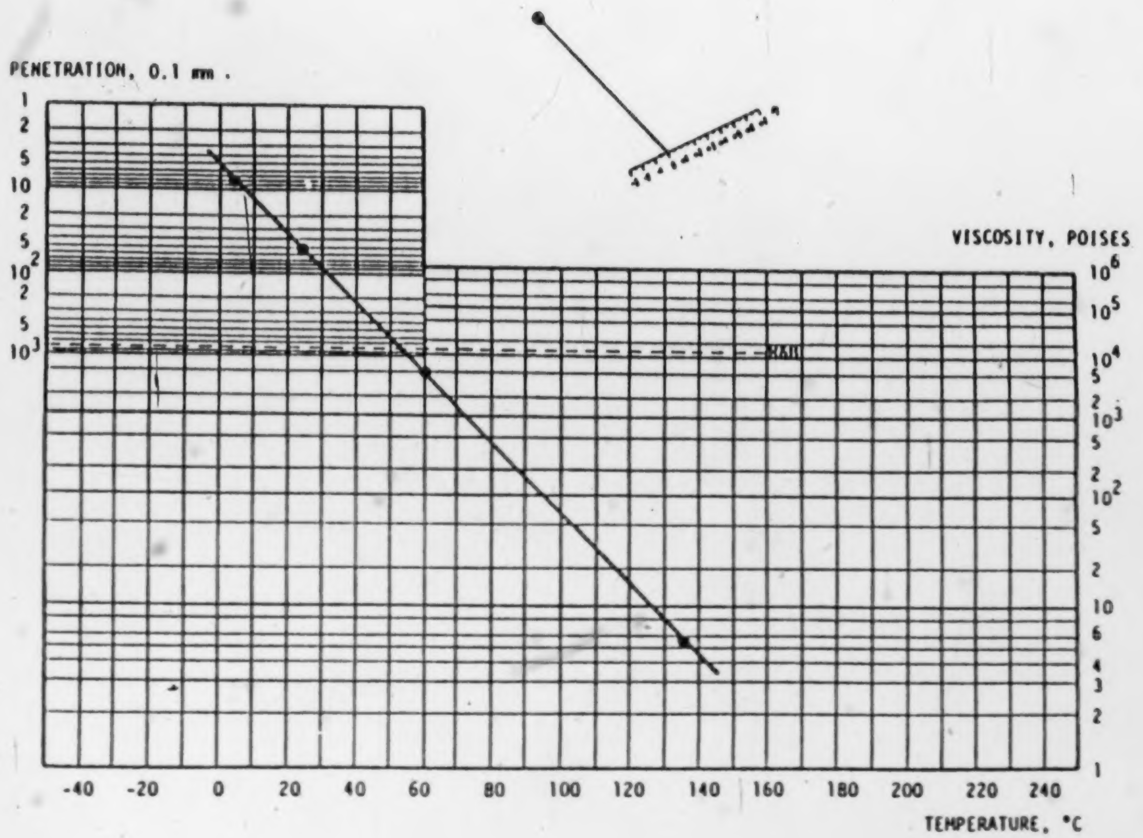


FIG. 6.8 THE BITUMEN TEST DATA CHART TO SHOW THE CHARACTERISTICS OF THE RECOVERED ASPHALT

TABLE 6.3 SUMMARY OF PROPERTIES OF THE OVERLAY MIX

Characteristics	Group 1	Group 2	Group 3	Group 4	Upper Course
Density (kg/m^3)	2196	2238	2248	2248	2280
Asphalt Content % by wt. of mix	-	5.3	5.0	4.9	4.9
Air Voids % of volume	-	7.0	7.0	7.1	5.8

Stiffness Modulus @7200 s of the Upper Course Overlay	C (F)	kPa (psi)
(Estimated by Bonnaure's Nomograph)	10 (50)	$1.3\text{x}10^{4}$ (2000)
	0 (32)	$7.6\text{x}10^{4}$ (11000)
	-10 (14)	$4.1\text{x}10^{5}$ (60000)
	-20 (-4)	$2.7\text{x}10^{6}$ (400000)
	-30 (-22)	$6.9\text{x}10^{6}$ (1000000)
	-40 (-40)	$1.8\text{x}10^{7}$ (2700000)

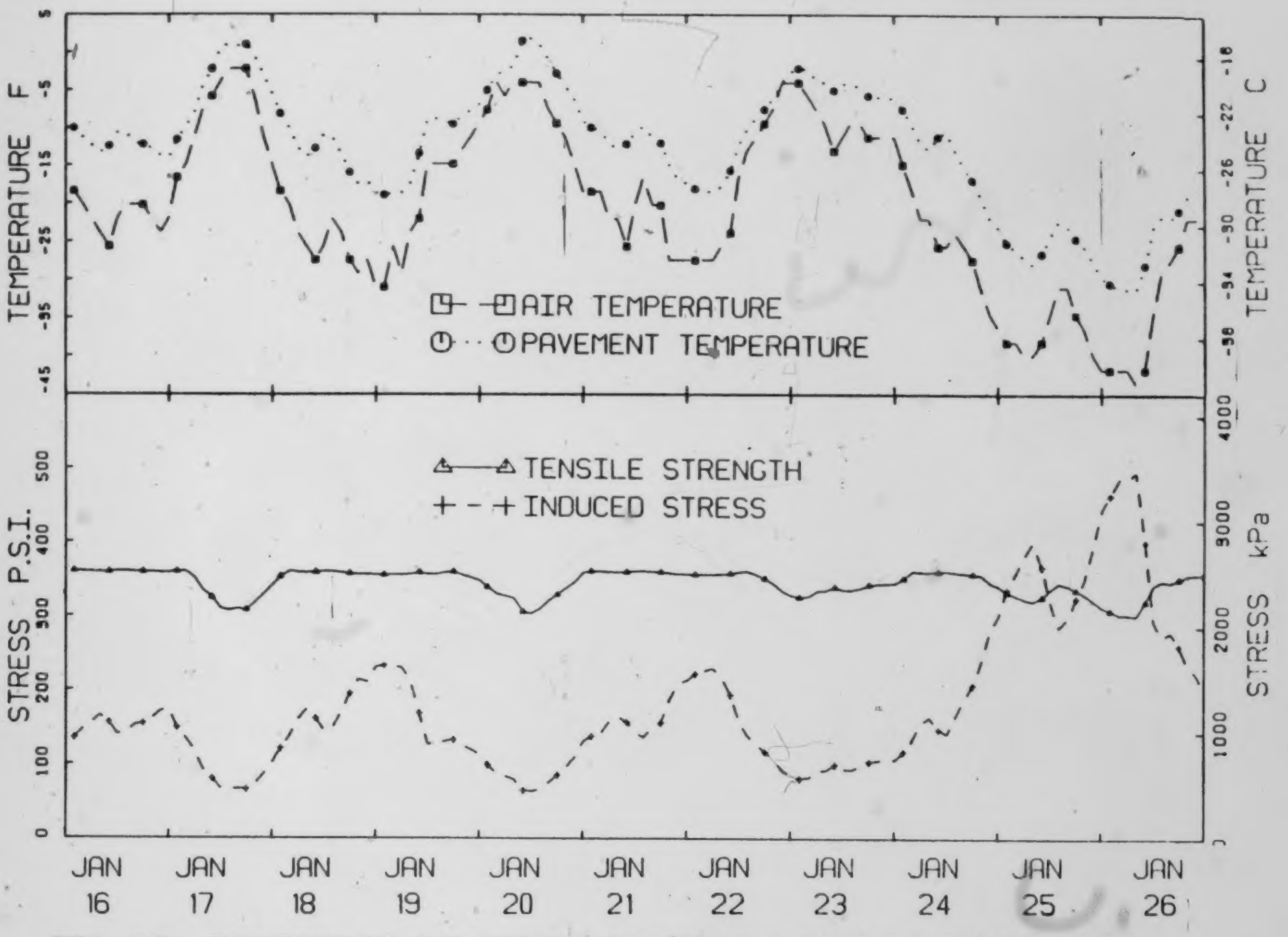


FIG. 6.9 TEMPERATURE AND STRESS CONDITIONS AT 1/2 INCH DEPTH OF THE OVERLAY BETWEEN JAN 16 TO JAN 26, 1984

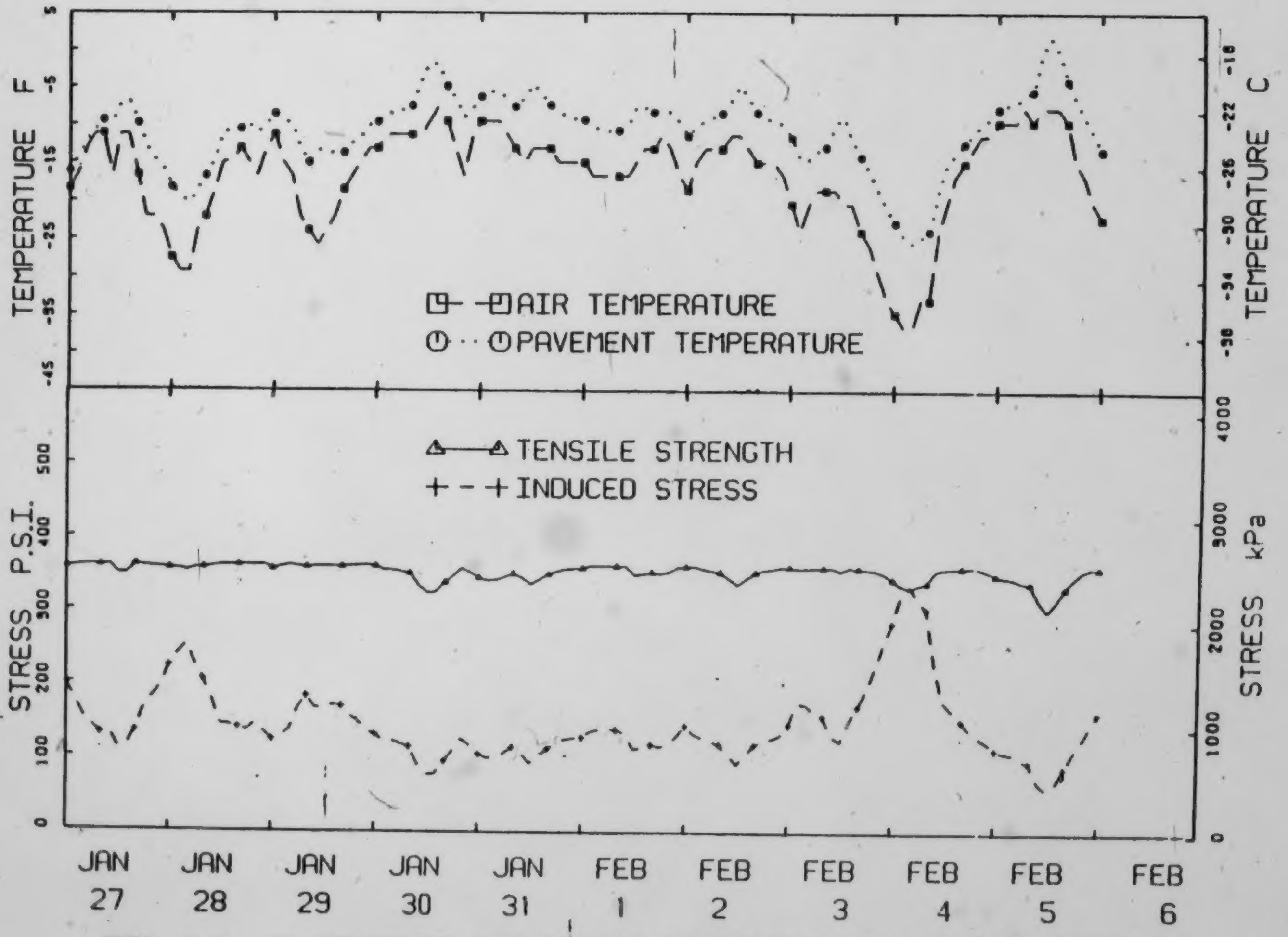


FIG. 6.10 TEMPERATURE AND STRESS CONDITIONS AT 1/2 INCH DEPTH OF THE OVERLAY BETWEEN JAN 27 TO FEB 5, 1984

CHAPTER 7

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary of the Literature Review

The purpose of this research was to study the reflection cracking problem in asphaltic concrete overlays of flexible airfield pavements in cold areas, mainly the reflection cracks caused by the thermal expansion and contraction of the pavement structure.

Various factors found from previous studies to be related to the reflection cracking problem were reviewed. Principal findings from these studies are:

1) The stiffness modulus of an asphaltic concrete is a primary factor of the fracture susceptibility of an overlay. Both the tensile strength and the induced thermal stress of an overlay are functions which depend on the stiffness modulus of the asphaltic concrete.
2) The stiffness modulus of an asphalt is a time and temperature dependent function. The effect of time and temperature on the stiffness modulus of asphalt can be superimposed one to another.

Recent experience with different rehabilitation methods has been collected and reviewed. Various degrees of success have been reported. Differences in performance may be caused by a) variability of roadbed performance, b) difference in climatic conditions, c) difference in roadbed preparation, d) presence of construction problems. Among these rehabilitation methods, it appears that a stress relieving interlayer using an open graded mix and an overlay mix using a low consistency asphalt or a modified asphalt perform better in inhibitating reflection cracking.

7.2 Conclusions From the Yellowknife Project

A test section constructed in Yellowknife in 1983 using geotextile Penroad as a stress relieving interlayer was studied. Cored samples were tested to obtain the tensile strength-temperature relationship of the asphaltic concrete and also to identify the properties of the asphalt in the mix. A computer analysis was used to predict the cracking potential of the overlay in the winter of 1983-1984.

The following conclusions are drawn from the construction phase of this experimental project:

1) The fabric selected for this project was not satisfactory. High plastic deformation of the fabric at low stresses caused installation difficulties. The fabric seams, resulting from sewing pieces of fabric together, were difficult to tack and they created sponginess which caused the overlay to crack. Problems arising from handling of the fabric affected the results.
2) The application of tack coat is an important procedure in the construction. The amount of tack coat applied, the type of tack coat used, and the length of curing time allowed are important factors to the effectiveness of the

fabric in preventing reflection cracking.

3) The fabric was successful in preventing the crack sealant from seeping through to the interface of the overlay and consequently prevented ridges forming in the overlay.
4) Installation of the fabric has shown no significant difference in retarding the reflection cracking. About 30 percent of the cracks reflected to the surface both in the fabric section and in the control section. Cracks are found at 5 metres intervals.
5) Experience gained from this project may be useful in improving fabric placement procedures.

The principal findings from the laboratory tests and the cracking potential analysis are as follows:

1) Consistency tests on recovered asphalt cement show that, despite the 150-200 pen asphalt used, the asphalt was hardened to that of a 40-60 pen asphalt, a retained penetration of 23 to 40 percent. Excessive hardening seems to have occurred during construction.
2) Using the COLD program to predict cracking by thermal shrinkage indicates that the overlay will crack when the ambient temperature drops below -38 C.
3) Predicting pavement cracking using the criteria of limiting stiffness of asphaltic mix indicates the cracking temperature to be about -52 C.
4) Review of temperature records shows that the minimum air temperature in the winter of 1983-1984 in Yellowknife was -42 C. There were twelve days in which the daily minimum

air temperature was below the computer predicted cracking temperature -38 C. Rate of temperature change could be as high as 3 C/hr (5 F/hr).

5) The analysis has considered the potential for cracking of the overlay under the thermal stresses alone. Stress concentration effect, as the result of the presence of existing cracks in the old pavement, and other factors such as the traffic loading, the base and subgrade movement may cause the actual cracking temperature higher than the predicted cracking temperature, and therefore higher potential for cracking.
6) The prediction of overlay cracking by the computer analysis is compatible with the observed field performance which shows 30 percent of the cracks reflected to the surface.

7.3 Recommendations

Recommendations arising from this investigation, based on literature reviews and the experience gained from the Yellowknife project are listed as follows:

1) Unless appropriate construction procedures are developed, fabrics with high plastic deformation at low stress should be avoided in future trials.
2) Future experiments on fabrics should avoid those formed by sewing pieces together.
3) The thermal stresses induced from the low temperatures which the pavement is expected to encounter in its

service life have to be considered in selecting the asphalt grade in future overlay designs. The COLD program provides a method to estimate the cracking potential of the resulting overlay.

4) Construction quality control procedures should be implemented to prevent excessive hardening of the asphalt.
5) Other rehabilitation methods such as an open graded stress relieving interlayer, a low consistency asphalt cement overlay or a modified asphalt overlay should also be considered.
6) Future studies related to the reflection cracking in cold areas should be directed toward the analytical study of the combined effect of thermal stress, stress concentration effect and base and subgrade effect on crack development.
7) Efforts should also be directed toward the estimation of crack frequency of the pavement.

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APPENDIX A

PROPERTIES OF "Mirafi" GEOTEXTILES

MIRAFI GEOTEXTILES P50

Nonwoven 100% Polyester

Physical Properties	Typical Values	
Mass (g/m^2)	200	5 a M 77
Specific Gravity	1.38	
Thickness (mm)	2.5	3.7 M 77
Tensile Strength (N)	420	9.2 M 77
Elongation At Break (%)	110	10.0 M 77
Mullen Burst Strength (kPa)	1200	11.1 M 77
Opening Size (μm)	150	CW 02215
Permeability (K cm s^{-1})	2.0×10^{-1}	

C.G.S.B. Test Methods Can 2 4.2 M 77

The information provided herein is believed to be accurate but must not be considered absolute. There are no guarantees nor representations on the part of the manufacturer, nor does it assume any liability or obligation in connection with the use, properties and/or suitability of the geotextiles described herein. IT IS THE SOLE RESPONSIBILITY OF THE USER TO PERFORM ALL NECESSARY TESTING TO DETERMINE THE SUITABILITY OF THE PRODUCTS FOR THE END-USE.

MIRAFI® IS THE REGISTERED TRADEMARK OF A COMPANY IN THE DOMINION TEXTILE GROUP

MIRAFI GEOTEXTILES P250

Nonwoven Polyester

Physical Properties	Typical Values	
Mass (g/m^2)	340	5 a M 77
Specific Gravity	1.38	
Thickness (mm)	3.3	3.7 M 77
Tensile Strength (N)	800	9.2 M 77
Elongation At Break (%)	100	10.0 M 77
Mullen Burst Strength (kPa)	2200	11.1 M 77
Opening Size (μm)	140	CW 02215
Permeability (K cm s^{-1})	2.0×10^{-1}	

C.G.S.B. Test Methods Can 2 4.2 M 77

The information provided herein is believed to be accurate but must not be considered absolute. There are no guarantees nor representations on the part of the manufacturer, nor does it assume any liability or obligation in connection with the use, properties and/or suitability of the geotextiles described herein. IT IS THE SOLE RESPONSIBILITY OF THE USER TO PERFORM ALL NECESSARY TESTING TO DETERMINE THE SUITABILITY OF THE PRODUCTS FOR THE END-USE.

MIRAFI® IS THE REGISTERED TRADEMARK OF A COMPANY IN THE DOMINION TEXTILE GROUP

APPENDIX B

DESCRIPTION OF THE TESTING APPARATUS

I. Controlled Temperature Chamber

The controlled temperature chamber shall be capable of maintaining test specimens at a constant temperature ±1 C within the range of 10 C to -30 C during the course of a test. A temperature monitoring device shall have its sensor embedded in a specimen of similar size and composition to the specimen which is to be tested and shall be capable of measuring temperature to ±0.5 C.

II. Loading Apparatus

Loading apparatus shall consist of the following:

1. Compression Testing Frame[1] ---- The compression frame shall have a minimum capacity of 45 kN (5 ton) and shall be capable of providing a nominal loading rate of 0.0254 mm/sec (0.06 in/min). The actual loading rate may vary from the nominal loading rate by ±10 percent but must be reproducible within ±1 percent.
2. Supplementary Bearing Bar or Plate ---- If the bearing face of the upper or lower loading block of the loading frame is less than the length of the cylinder to be tested, a supplementary bearing bar or plate shall be used. The supplementary bearing bar shall conform to the specification for this item in the Standard Test Method for Splitting Tensile

---

[1] A suitable device (Wykeham Farrance Mod. 57, 5 ton compression tester) may be obtained from Wykeham Farrance Engineering, Ltd., 127 Edinburgh Avenue, Slough, Bucks, U.K.

Strength of Cylindrical Concrete Specimens (ASTM Designation: C 496-85), except that the width of the bearing bar or plate shall be at least 33 mm.

3. Bearing Strips ---- A pair of 12.7 mm wide steel bearing strips that are curved at the interface with the specimen and have a radius equal to that of the specimen shall be used. The bearing strips shall be placed between the specimen and both the upper and lower bearing blocks of the testing machine or between the specimen and the supplemental bars or plates, if used (see Section II.2).

III. Loading Measurment Apparatus

The loading measurement apparatus shall consist of the following:

1. Load Cell[2] ---- The load cell shall have a minimum capacity of 45 kN (5 ton) and shall be capable of measuring compressive loading to ±1 percent of true at the rate of loading prescribed in Section II.1.

IV. Gauge Points, and Marking and Mounting Apparatus

Gauge points, and marking and mounting apparatus shall consist of the following:

1. Gauge Points ---- The gauge points shall be 9.53 mm x 9.53 mm x 4.76 mm (±0.025 mm from mean in any dimension) brass paltes.
2. Gauge Point Jig ---- The gauge point jig shall

---

[2] A suitable device (Kwoya Musen Load Cell Mod. LC-5, 5 ton) may be obtained from Kwoya Musen Kenkyujo Co., Ltd., Tokyo, Japan.

provide slots for marking the specimen and holes for mounting the gauge points (Fig. A1 of Anderson and Hahn, 1968)

V. Deformation Measurement Apparatus

Deformation measurement apparatus shall consist of the following:

1. Horizontal Displacement Gauges[3] ---- The displacement gauges shall be two A.C. type Linear Variable Differential Transducers of matched sensitivity (within 5%) and be capable of measuring displacements to within ±0.00125 mm, and shall have a stroke of not less than ±0.25 mm
2. Horizontal Displacement Gauge Core and Coil Assemblies ---- The two displacement gauge core and coil assemblies which hold the displacement gauges shall be made of brass (Fig. A2 of Anderson and Hahn, 1968)
3. Displacement Gauge Calibration Jig ---- The displacement gauge calibration jig shall be made of brass and aluminum (Fig. A3 of Anderson and Hahn, 1968). The dial gauge which comprises a portion of the displacement gauge calibration jig shall be a 0.000254 mm (0.00001 in) dial gauge.
4. Vertical Displacement Gauge[4] ---- The displacement

---

[3] Suitable devices (Sanborn Linear Variable Differential Transducers Mod. 595 DT 025) may be obtained from the Sanborn Co. 175 Wyman Street, Waltham 54, Massachusetts, U.S.A.

[4] A suitable device (Hewlett Packard Linear Variable Differential Transducer Mod. 7DC.Dt-250) may be obtained

gauge shall be a D.C. type Linear Variable Differential Transducer and be capable of measuring displacements to within ±0.01 mm, and shall have a stroke of not less than ±5 mm.

Fig.B.1 shows the various deformation measurement apparatus and their assemblies.

VI. Data Acquisition Apparatus

1. Signal Conditioner ---- The signal conditioner shall be a device which is capable of performing the following functions:

   a) provides excitations to the load cell, A.C. type LVDTs and D.C. type LVDT.

   b) sums two signals.

   c) balances signals to give an electrical zero at the start of tests.

   d) amplifies output signals.

   e) transmits output signals to a recorder.

   Fig. B.2 shows a functional sketch of the signal conditioner.

2. Recorder[5] ---The recorder shall be a multiple channels X-Y recorder which is capable of providing trace curves of the applying load, horizontal deformations at the two faces, average horizontal deformation and vertical deformation from the output signals.

---

[4] (cont'd) from the Hewlett Packard Co. 175 Wyman St., Walham, Massachusetts, U.S.A. 02154.

[5] A suitable device (Servogor 460, a six channels X-Y recorder) may be obtained from the BBC - Metrawatt/ Goerz Co. Raritan Center, 165 Fieldcrest Ave., Edison, New Jersey U.S.A. 08817.

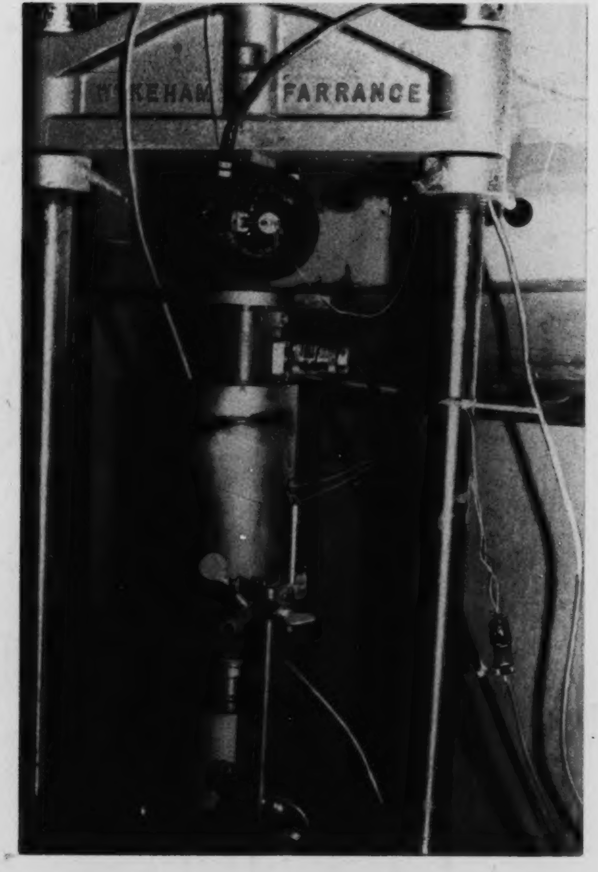

FIG. B.1 THE DEFORMATION MEASUREMENT APPARATUS AND THEIR ASSEMBLIES

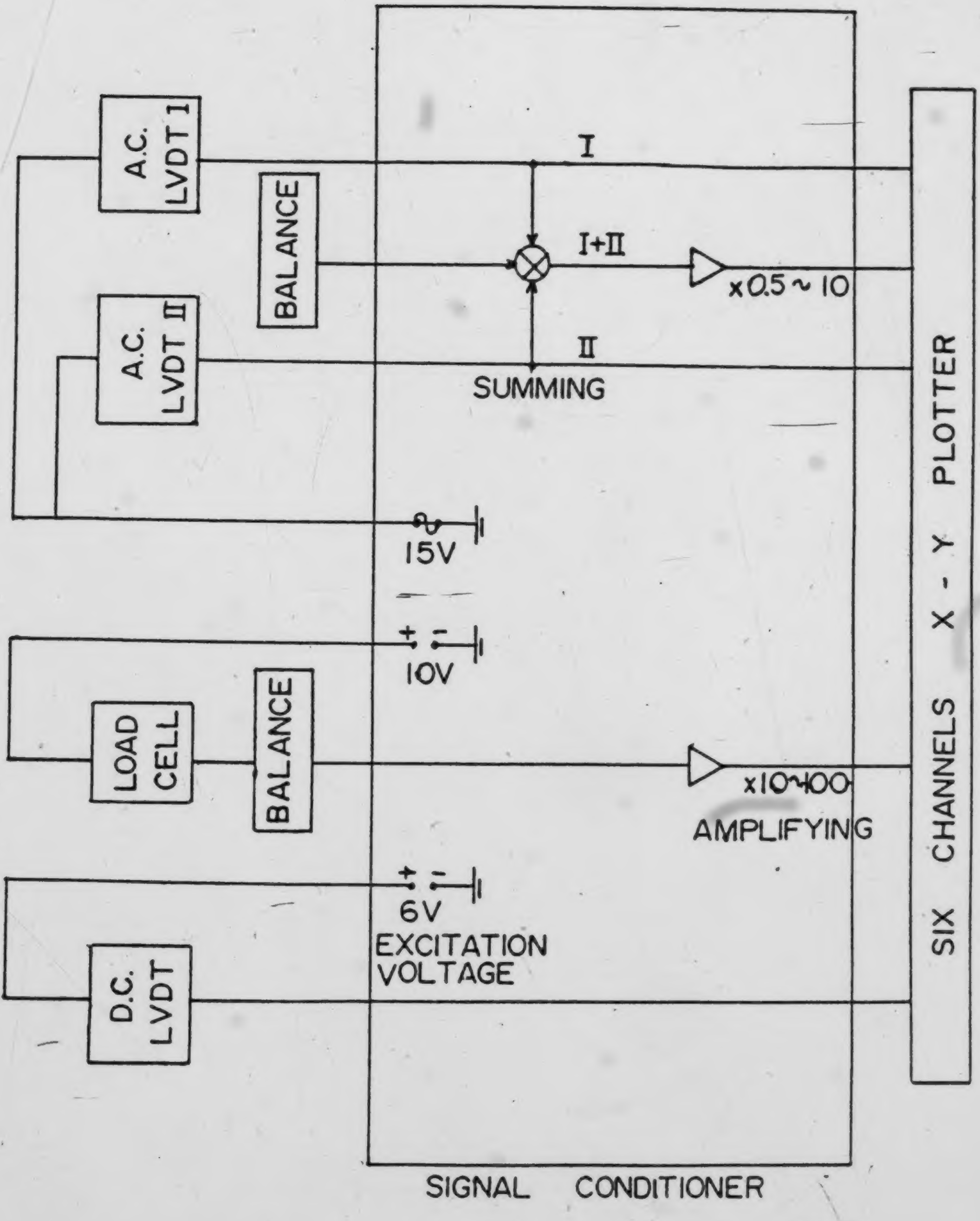

FIG. B.2 A FUNCTIONAL SKETCH OF THE SIGNAL CONDITIONER

APPENDIX C

TEMPERATURE AND RADIATION DATA
BETWEEN JAN 16 AND FEB 5, 1984

```
JAN 16,1984 HOURLY TEMPERATURES (F)
                                 -16.6   -18.4   -18.4   -20.2   -20.2   -20.2   -22.0   -22.0   -23.8
                                 -23.8   -25.6   -25.6   -22.0   -20.2   -20.2   -20.2   -20.2   -20.2
                                 -20.2   -20.2   -22.0   -22.0   -23.8   -23.8   -22.0

            HOURLY RADIATIONS (LANGLEYS)
                                   0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.5
                                   3.0     7.5    10.8    10.1     7.6     3.3     0.5     0.0     0.0
                                   0.0     0.0     0.0     0.0     0.0     0.0     0.0


JAN 17,1984 HOURLY TEMPERATURES (F)
                                 -22.0   -18.4   -16.6   -14.8   -14.8   -13.0   -11.2    -9.4    -7.6
                                  -7.6    -5.8    -4.0    -4.0    -2.2    -2.2    -2.2    -2.2    -2.2
                                  -2.2    -5.8    -5.8    -9.4   -11.2   -13.0   -14.8

            HOURLY RADIATIONS (LANGLEYS)
                                   0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     3.8
                                   2.2     4.6     6.5     6.5     3.9     2.0     0.4     0.0     0.0
                                   0.0     0.0     0.0     0.0     0.0     0.0     0.0


JAN 18,1984 HOURLY TEMPERATURES (F)
                                 -14.8   -16.6   -18.4   -20.2   -20.2   -23.8   -23.8   -23.8   -25.6
                                 -27.4   -27.4   -27.4   -25.6   -23.8   -22.0   -22.0   -23.8   -25.6
                                 -27.4   -27.4   -29.2   -25.6   -27.4   -27.4   -31.0

            HOURLY RADIATIONS (LANGLEYS)
                                   0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.5
                                   4.6    10.0    13.0    13.4    11.0     5.8     0.6     0.0     0.0
                                   0.0     0.0     0.0     0.0     0.0     0.0     0.0


JAN 19,1984 HOURLY TEMPERATURES (F)
                                 -31.0   -25.6   -31.0   -31.0   -25.6   -25.6   -29.2   -27.4   -23.8
                                 -23.8   -22.0   -20.2   -14.8   -14.8   -14.8   -13.0   -14.8   -14.8
                                 -14.8   -13.0   -13.0   -13.0   -11.2   -13.0    -9.4

            HOURLY RADIATIONS (LANGLEYS)
                                   0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.8
                                   3.9     8.6    11.3    10.7     8.3     3.1     0.5     0.0     0.0
                                   0.0     0.0     0.0     0.0     0.0     0.0     0.0
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JAN 20,1984 HOURLY TEMPERATURES (F)
                                -9.4    -7.6    -7.6    -5.8    -4.0    -4.0    -5.8    -5.8    -4.0
                                -4.0    -4.0    -4.0    -4.0    -4.0    -4.0    -4.0    -7.6    -7.6
                                -9.4   -11.2   -11.2   -13.0   -14.8   -18.4   -18.4

            HOURLY RADIATIONS (LANGLEYS)
                                 0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.8
                                 4.4     8.8     9.0     6.6     6.1     2.9     0.6     0.0     0.0
                                 0.0     0.0     0.0     0.0     0.0     0.0     0.0

JAN 21,1984 HOURLY TEMPERATURES (F)
                               -18.4   -18.4   -18.4   -18.4   -18.4   -20.2   -22.0   -22.0   -22.0
                               -23.8   -25.6   -25.6   -20.2   -18.4   -16.6   -16.6   -20.2   -18.4
                               -20.2   -22.0   -27.4   -27.4   -27.4   -25.6   -27.4

            HOURLY RADIATIONS (LANGLEYS)
                                 0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.8
                                 3.7     8.5    12.1     8.7     7.3     2.8     0.6     0.0     0.0
                                 0.0     0.0     0.0     0.0     0.0     0.0     0.0

JAN 22,1984 HOURLY TEMPERATURES (F)
                               -27.4   -27.4   -27.4   -27.4   -27.4   -27.4   -27.4   -25.6   -25.6
                               -23.8   -23.8   -20.2   -16.6   -14.8   -13.0   -11.2   -11.2   -11.2
                                -9.4    -9.4    -7.6    -5.8    -4.0    -4.0    -4.0

            HOURLY RADIATIONS (LANGLEYS)
                                 0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.6
                                 2.2     3.7     4.3     3.9     2.6     1.2     0.3     0.0     0.0
                                 0.0     0.0     0.0     0.0     0.0     0.0     0.0

JAN 23,1984 HOURLY TEMPERATURES (F)
                                -4.0    -4.0    -4.0    -4.0    -5.8    -9.4    -7.6    -7.6    -9.4
                               -13.0   -13.0   -13.0   -11.2    -9.4    -9.4    -9.4    -9.4   -11.2
                               -11.2   -11.2   -11.2   -11.2   -11.2   -11.2   -11.2

            HOURLY RADIATIONS (LANGLEYS)
                                 0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.8
                                 2.3     4.3     6.0     6.0     3.8     1.8     0.5     0.0     0.0
                                 0.0     0.0     0.0     0.0     0.0     0.0     0.0
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JAN 24,1984 HOURLY TEMPERATURES (F)
                                -11.2    -13.0    -14.8    -16.6    -18.4    -20.2    -22.0    -22.0    -22.0
                                -22.0    -25.6    -25.6    -25.6    -23.8    -23.8    -23.8    -25.6    -25.6
                                -27.4    -29.2    -31.0    -34.6    -34.6    -32.8    -36.4

            HOURLY RADIATIONS (LANGLEYS)
                                  0.0      0.0      0.0      0.0      0.0      0.0      0.0      0.0      1.1
                                  5.4     11.0     14.0     14.4      8.8      3.6      0.9      0.0      0.0
                                  0.0      0.0      0.0      0.0      0.0      0.0      0.0

JAN 25,1984 HOURLY TEMPERATURES (F)
                                -36.4    -36.4    -38.2    -38.2    -38.2    -38.2    -40.0    -40.0    -40.0
                                -38.2    -38.2    -36.4    -34.6    -32.8    -31.0    -31.0    -31.0    -32.8
                                -34.6    -34.6    -36.4    -38.2    -40.0    -41.8    -41.8

            HOURLY RADIATIONS (LANGLEYS)
                                  0.0      0.0      0.0      0.0      0.0      0.0      0.0      0.0      0.6
                                  1.8      3.5      5.1      5.9      6.1      4.8      1.2      0.0      0.0
                                  0.0      0.0      0.0      0.0      0.0      0.0      0.0

JAN 26,1984 HOURLY TEMPERATURES (F)
                                -41.8    -41.8    -41.8    -43.6    -41.8    -41.8    -41.8    -41.8    -43.6
                                -41.8    -41.8    -38.2    -34.6    -31.0    -29.2    -27.4    -27.4    -25.6
                                -25.6    -23.8    -22.0    -22.0    -22.0    -20.2    -18.4

            HOURLY RADIATIONS (LANGLEYS)
                                  0.0      0.0      0.0      0.0      0.0      0.0      0.0      0.0      1.4
                                  5.5     11.3     14.7     15.7     13.4      5.8      1.1      0.0      0.0
                                  0.0      0.0      0.0      0.0      0.0      0.0      0.0

JAN 27,1984 HOURLY TEMPERATURES (F)
                                -18.4    -16.6    -16.6    -13.0    -13.0    -13.0    -11.2    -11.2    -11.2
                                -14.8    -16.6    -14.8    -11.2    -11.2    -11.2    -13.0    -16.6    -18.4
                                -22.0    -22.0    -22.0    -23.8    -23.8    -27.4    -27.4

            HOURLY RADIATIONS (LANGLEYS)
                                  0.0      0.0      0.0      0.0      0.0      0.0      0.0      0.0      0.6
                                  2.7      5.3      7.1      7.8      6.5      3.0      1.1      0.0      0.0
                                  0.0      0.0      0.0      0.0      0.0      0.0      0.0
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JAN 28,1984 HOURLY TEMPERATURES (F)
                                  -27.4   -27.4   -29.2   -29.2   -29.2   -25.6   -23.8   -22.0   -22.0
                                  -20.2   -18.4   -16.6   -14.8   -14.8   -14.8   -14.8   -13.0   -11.2
                                  -14.8   -14.8   -16.6   -13.0   -13.0   -11.2   -11.2

            HOURLY RADIATIONS (LANGLEYS)
                                    0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.5
                                    1.5     2.6     4.5     5.2     3.7     1.3     0.4     0.0     0.0
                                    0.0     0.0     0.0     0.0     0.0     0.0     0.0

JAN 29,1984 HOURLY TEMPERATURES (F)
                                  -11.2   -11.2   -14.8   -13.0   -16.6   -20.2   -22.0   -23.8   -23.8
                                  -25.6   -25.6   -25.6   -23.8   -22.0   -22.0   -20.2   -18.4   -16.6
                                  -16.6   -16.6   -14.8   -13.0   -13.0   -13.0   -13.0

            HOURLY RADIATIONS (LANGLEYS)
                                    0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     1.6
                                    7.0    10.6    10.3     7.9     6.3     3.1     0.6     0.0     0.0
                                    0.0     0.0     0.0     0.0     0.0     0.0     0.0

JAN 30,1984 HOURLY TEMPERATURES (F)
                                  -13.0   -11.2   -11.2   -13.0   -11.2   -11.2   -11.2   -13.0   -11.2
                                   -9.4   -11.2   -13.0    -9.4    -7.6    -7.6    -9.4    -9.4   -11.2
                                  -13.0   -14.8   -16.6   -13.0   -11.2    -9.4    -9.4

            HOURLY RADIATIONS (LANGLEYS)
                                    0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     2.1
                                    5.6    10.5    14.2    14.2    11.5     6.2     1.3     0.0     0.0
                                    0.0     0.0     0.0     0.0     0.0     0.0     0.0

JAN 31,1984 HOURLY TEMPERATURES (F)
                                   -9.4    -7.6    -9.4    -9.4    -9.4   -11.2   -11.2   -14.8   -13.0
                                  -14.8   -14.8   -14.8   -13.0   -14.8   -13.0   -13.0   -13.0   -13.0
                                  -14.8   -14.8   -14.8   -14.8   -14.8   -14.8   -14.8

            HOURLY RADIATIONS (LANGLEYS)
                                    0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.1     1.5
                                    3.8     7.7    11.1    10.4     8.6     4.7     1.6     0.2     0.0
                                    0.0     0.0     0.0     0.0     0.0     0.0     0.0
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FEB 1,1984  HOURLY TEMPERATURES (F)
                          -14.8   -16.6   -16.6   -16.6   -16.6   -16.6   -16.6   -16.6   -16.6
                          -16.6   -16.6   -14.8   -14.8   -13.0   -13.0   -13.0   -13.0   -13.0
                          -11.2   -13.0   -13.0   -14.8   -16.6   -18.4   -18.4

            HOURLY RADIATIONS (LANGLEYS)
                            0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.1     0.7
                            1.8     2.8     5.3     7.3     5.4     2.0     1.0     0.1     0.0
                            0.0     0.0     0.0     0.0     0.0     0.0     0.0

FEB 2,1984  HOURLY TEMPERATURES (F)
                          -18.4   -14.8   -14.8   -14.8   -13.0   -13.0   -13.0   -13.0   -13.0
                          -13.0   -11.2   -13.0   -11.2   -13.0   -13.0   -13.0   -14.8   -14.8
                          -14.8   -14.8   -14.8   -14.8   -16.6   -16.6   -20.2

            HOURLY RADIATIONS (LANGLEYS)
                            0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.8
                            2.9     4.4    10.0     7.8     2.9     3.3     1.0     0.0     0.0
                            0.0     0.0     0.0     0.0     0.0     0.0     0.0

FEB 3,1984  HOURLY TEMPERATURES (F)
                          -20.2   -23.8   -23.8   -23.8   -20.2   -20.2   -18.4   -20.2   -18.4
                          -18.4   -18.4   -18.4   -20.2   -20.2   -20.2   -20.2   -23.8   -23.8
                          -25.6   -27.4   -29.2   -31.0   -32.8   -32.8   -34.6

            HOURLY RADIATIONS (LANGLEYS)
                            0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.1     1.3
                            4.7     9.1     9.3    11.6     6.0     2.1     0.8     0.1     0.0
                            0.0     0.0     0.0     0.0     0.0     0.0     0.0

FEB 4,1984  HOURLY TEMPERATURES (F)
                          -34.6   -36.4   -36.4   -36.4   -36.4   -34.6   -32.8   -32.8   -32.8
                          -27.4   -23.8   -22.0   -20.2   -18.4   -16.6   -14.8   -14.8   -13.0
                          -13.0   -13.0   -11.2   -11.2   -11.2    -9.4    -9.4

            HOURLY RADIATIONS (LANGLEYS)
                            0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.1     1.3
                            3.8     6.6     7.8     6.5     4.4     1.5     0.2     0.1     0.0
                            0.0     0.0     0.0     0.0     0.0     0.0     0.0
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FEB 5,1984  HOURLY TEMPERATURES (F)
                            -9.4     -9.4     -9.4     -9.4     -9.4     -7.6     -7.6     -9.4     -9.4
                            -7.6     -7.6     -7.6     -7.6     -7.6     -7.6     -9.4     -9.4    -11.2
                           -14.8    -14.8    -16.6    -18.4    -20.2    -22.0    -22.0

            HOURLY RADIATIONS (LANGLEYS)
                             0.0      0.0      0.0      0.0      0.0      0.0      0.0      0.1      3.1
                            10.2     14.7     16.9     16.9     13.7      8.5      2.7      0.1      0.0
                             0.0      0.0      0.0      0.0      0.0      0.0      0.0
```

APPENDIX D

SAMPLE CALCULATIONS OF AIR VOIDS IN OVERLAY

The following data about the properties of the upper course of the overlay are obtained from the engineer's report of the project.

Density of mix = 2280 kg/m^3
Asphalt content = 4.9 % by weight of mix
Bulk specific gravity of combined aggregrate = 2.602
Bitumen specific gravity = 1.03

For 1000 cc volume of mix,

the weight of mix = 2280 g

the weight of asphalt = $2280 \times \frac{4.9}{100} = 111.72$ g

then the volume of asphalt = $\frac{111.72}{1.03} = 108.47$ cc

the weight of aggregates = 2280 - 111.72 = 2168.28 g

then the volume of aggregates = $\frac{2168.28}{2.602} = 833.31$ cc

the volume of air = 1000 - 833.31 - 108.47 = 58.22 cc

the percentage of air voids = $\frac{58.22}{1000} = 5.8$ %